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COLUMBIA RIVER
MURPHY CREEK PROJECT
PRELIMINARY DESIGN STUDY
PHASE 1

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SYNOPSIS

The proposed Murphy Creek Project would be located about 3 km upstream of the City of Trail on the Columbia River and would develop an average gross head of about 15 m between the project site and the Hugh Keenleyside Dam some 34 km upstream. The Hugh Keenleyside Dam, completed in late 1968, was constructed to provide storage (Arrow Lakes Reservoir) under the terms of the Columbia River Treaty.

Preliminary Design studies for the Murphy Creek Project and the proposed Keenleyside Powerplant Project were conducted concurrently because the reservoir level selected for the Murphy Creek Project would directly affect the tailwater level and generating capability at Keenleyside. For a combined Keenleyside-Murphy Project, the recommended reservoir level for the Murphy Creek Project is El. 424 m. The two projects would each develop about half of the total head of 31 m.

Because of the considerable drawdown of the Arrow Lakes Reservoir with current operation under the Treaty, studies were made of several alternative operating regimes. The regimes were representative of both present conditions and of long-term conditions when expiry or modification of the Treaty would permit reduced drawdown, increased generation at both Keenleyside and Murphy Creek and reduced environmental impacts. An operating regime that would reflect these benefits has been adopted for project design.

The Murphy Creek Project would consist of a 27 m high embankment dam across the river with a powerhouse comprising five 60 MW Kaplan units (based on 1982 economic criteria), located on the right bank of the river. A five-bay gated overflow spillway would be between the powerhouse and the dam.

Investigations have shown that the foundations are competent to support the proposed structures and that suitable construction materials are available locally from required excavations and borrow pits. The concrete spillway and powerhouse structures would be founded on rock while the embankment dam would be founded on river gravels. A concrete cut-off wall would extend to rock at about El. 335 m beneath the embankment dam and abutment and for a distance of about 270 m beyond existing ground surface into the left abutment.

Studies of the reservoir shoreline have indicated that beaching due to wave action and regression would occur following reservoir filling. Siltation would not affect the life of the reservoir. Portions of the shoreline would be protected against erosion due to the reservoir. The possibility of deep-seated sliding due to reservoir filling and normal drawdown is considered remote.

Flows, capacity, energy and cost data for the Murphy Creek Project are given in the following tabulation:

Average River Flow (1940-75)	2050 m^3/s
Installed Generating Capacity	300 MW
Number of Units	5
Average Annual Energy*	1970 GW.h
Construction Cost (1982 prices)**	$632 million
Construction Cost of Capacity	$2107/kW

Note: * Including an allowance for losses at Brilliant of 145 GW.h.

** Excluding inflation, interest during construction and corporate overhead.

Based on B.C. Hydro's 1983 Electrical System Plan, the first unit would be in-service in July 1993 with the remaining units following at intervals of about 4 months; the last unit (Unit 5) would come in-service in October 1994. The overall construction period to first in-service would be 5 1/2 years.

REPORT

SECTION 1.0 - INTRODUCTION

1.1 BACKGROUND

The Murphy Creek Project would be located on the Columbia River about 3 km upstream of the City of Trail, 23 km downstream of the confluence of the Columbia and Kootenay rivers and 34 km downstream of the Hugh Keenleyside Dam on the Columbia River (Figs. 1-1 and 1-2).

Prior to ratification of the Columbia River Treaty in 1964, about 110 m of head had been developed in the lower reach of the Kootenay River through six hydroelectric projects with a total installed capacity of 275 MW. Four of these projects are owned by Cominco Ltd., one is owned by West Kootenay Power and Light Ltd., and one by the City of Nelson. The location of these plants and the other hydroelectric plants located in the Columbia River basin are shown on Fig. 1-1.

Under the terms of the Columbia River Treaty between Canada and the United States, the Mica and Hugh Keenleyside dams were constructed on the Columbia River in Canada. Also under the Treaty, Duncan Dam was constructed on the Duncan River, a tributary of Kootenay Lake; in the United States the Libby Dam was constructed on the Kootenai River. Other major hydroelectric projects in the Columbia River basin in Canada include the Kootenay Canal Project, the Seven Mile Project, and the Revelstoke Project, scheduled for completion in 1984.

Significant studies for the Murphy Creek Project include the 1972 Energy Board Studies,[1] 1978 Prefeasibility Studies[2] and 1981 Feasibility Studies.[3]

Feasibility studies were completed in 1981 and, because of the considerable interdependence of Murphy and Keenleyside, a prefeasibility level study for Keenleyside[4] was undertaken to permit a combined optimization of both projects.[5,6] Alternative Murphy reservoir levels (as measured at Castlegar Ferry) of Els. 424, 427 and 430 m were investigated. These alternative reservoir levels would have varying affects on the potential generating capacity at Keenleyside as well as on Cominco's Brilliant plant on the Kootenay River. An economic evaluation of these alternative levels was undertaken.[7] Based on the results of these studies and an assessment of project impacts it was judged that the incremental benefits of the higher Murphy reservoir level were outweighed by the incremental regional and environmental costs of the greater flooding.[8] Consequently, a Keenleyside-Murphy development with a reservoir level of El. 424 m was selected as the optimal alternative development and the basis for further studies.[5,6]

Phase 1 Preliminary Design studies for the Murphy Creek Project were begun in 1981 and are documented in this report. Similar studies for the Keenleyside Powerplant Project have also been completed.[9]

1.2 ASSIGNMENT

Under Assignment 480-143 dated 17 February 1981 and revised 10 August 1981, the Hydroelectric Generation Projects Division was authorized to provide engineering services for a preliminary design study of the Murphy Creek Project. The work was to include:

1. Geotechnical investigations.

2. Engineering services for preliminary design in two phases. Phase 1 was to include all design work required for project licensing including a selected project description, hydraulic and hydrology studies, reservoir bank stability studies, flowage studies (including engineering studies required for any suggested shoreline

improvements), construction facilities studies and navigation and fish passage studies.

Phase 2 was to include all other preliminary design studies required prior to final design including hydraulic model studies.

3. Preparation of the following reports:

   Engineering Description
   Preliminary Design - Phase 1
   Preliminary Design - Phase 2

1.3 SCOPE OF WORK

The work described in this report is for Phase 1 of the Preliminary Design Assignment and included field investigations and office studies sufficient to select a project arrangement and to permit decisions to be reached on the following items:

1. Project arrangement.

2. Type of embankment dam.

3. Type of foundation seepage control.

4. Type and capacity of spillway.

5. Type and tentative capacity of river diversion during construction.

6. Reservoir and tailwater levels.

7. Powerhouse arrangement, type and number of units.

8. Type and extent of protective works for the reservoir shoreline.

9. Construction access roads and construction facility areas.

10. Relocation of highway, railway and transmission lines around project site.

The main work carried out relative to the above items are listed below:

1. Review of information available from previous studies.

2. Preparation of topographic mapping for:

   a. The reservoir area at a scale of 1:5000.

   b. The reservoir area at a scale of 1:2500.

   c. The railway relocation area at a scale of 1:1250.

3. Geotechnical investigations in the damsite area consisting of geological mapping, about 2336 m of drilling, bulldozer trenching, test pitting and seismic refraction surveys.

4. Construction material investigations for concrete aggregate and impervious fill including laboratory testing.

5. Geotechnical investigations for reservoir shoreline assessment studies consisting of airphoto interpretation, site reconnaissance, surveys and exploratory drilling of overburden.

6. Hydraulic field investigations including river gauging and measuring of river channel cross sections throughout the reservoir.

7. Hydrometeorological studies to determine discharge capacities for the spillway and construction diversion facilities including derivation of the Probable Maximum Flood, Spillway Design Flood and Diversion Design Flood.

8. Design and cost estimates for alternative unit types including vertical shaft Kaplan turbines and horizontal-shaft bulb units. Selection of one alternative for further refinement.

9. Design and cost estimates for alternative project arrangements; selection of one alternative for further refinement.

10. Design, layouts and cost estimates for three alternative reservoir levels for the downstream Murphy Creek Project; Els. 421, 424 and 427. For all three levels, vertical shaft Kaplan turbines were assumed. The cost estimates were prepared for use in the benefit/cost study[8] to review the selection of El. 424 as the nominal reservoir level for the Murphy Creek Project.

11. Computer simulated power studies for the three alternative Murphy reservoir levels and for two different operating regimes for the Arrow Lakes Reservoir. Regime 1 generally represents existing operation and Regime 2 represents future operation with reduced reservoir drawdown and significantly increased generation potential.

12. Optimization studies for the selection of installed generating capacity.

13. Preparation of a preliminary design layout for the selected project arrangement.

14. Studies of alternative construction access routes.

15. Relocation studies for Provincial Highway 22, Canadian Pacific Railway (CPR) and West Kootenay Power & Light (WKPL) transmission lines, around the project site.

16. Relocation studies for the CPR line along the reservoir shoreline.

17. Studies of reservoir impact on municipal infrastructure.

18. Studies of reservoir impact on BC Timber Co. Ltd. facilities.

19. Studies to select a location for and preliminary layout of a construction camp.

20. Assessment of groundwater conditions in the reservoir area.

21. Assessment of the impact of noise, vibration and dust effects from construction activities on adjacent communities.

22. Preparation of a construction schedule and estimate of construction costs for the selected project arrangement.

23. Preparation of an Engineering Description.

24. Preparation of this report on the Phase 1 Preliminary Design Study.

SECTION 2.0 - PROJECT AREA

2.1 AREA DESCRIPTION, LOCATION AND ACCESS

The Columbia River drainage basin (Fig. 1-1) located in southeastern British Columbia to the west of the Rocky Mountains, is some 500 km inland from the Coast Mountains. From its source, Columbia Lake, the Columbia River flows about 330 km northwest along the Rocky Mountain Trench to Kinbasket Lake Reservoir, the impoundment for the Mica Project. From Mica, the river flows almost due south for about 450 km through the Selkirk Trench past the Revelstoke (under construction) and Hugh Keenleyside dams to reach the international boundary. From the boundary the river continues through the United States to the Pacific Ocean.

The Kootenay River, the major tributary to the Columbia River upstream of the Murphy Creek Project, originates in the Beaverfoot range of the Rocky Mountains some 150 km north of Columbia Lake and then flows in a southeasterly direction before entering the Rocky Mountain Trench near Canal Flats. The Kootenay River continues in a southeasterly direction for 150 km to the international boundary and makes a 260 km loop through Montana and Idaho, where it is known as the Kootenai. The Libby Dam is located on the Kootenai River in Montana. The river re-enters Canada near the south end of Kootenay Lake. From this lake, the river flows west for about 30 km before joining the Columbia River at Castlegar. The Corra Linn, City of Nelson, Upper Bonnington, Lower Bonnington, Kootenay Canal, South Slocan and Brilliant projects are located on this 30 km reach of the Kootenay River.

The Columbia River drainage area above the Murphy Creek Project site is characterized by wide glaciated valleys and high mountain chains which cross the region along a northwest-southeast axis. The drainage basin to the east is bounded by the Rocky Mountains. The western edge of the drainage basin is formed by the Monashee Mountains and by the Columbia Mountains. Running parallel between the Monashee Mountains and the Rocky Mountains are the Selkirk Mountains which separate the Arrow Lakes valley from the Kootenay Lake valley, and the Purcell Mountains which separate the Kootenay Lake valley from the Rocky Mountain Trench. The mountains tend to be lower in the south, but the ranges all contain peaks up to El. 3000 m, and each mountain chain forms a significant elevation barrier to the prevailing weather systems coming from the Pacific Ocean in the west.

The Columbia valley floor rises from about 425 m above sea level near Trail to 800 m near Columbia Lake. Upstream of Castlegar approximately 40 percent of the Columbia drainage basin is above El. 1800 m whereas the Kootenay drainage basin is lower, with approximately 30 percent of the area above this elevation.

Most of the Columbia basin remains in its virgin forested state. Dense forest vegetation thins above 1500 m and tree lines lie generally in the El. 1800 to 2100 m range. Past glaciation of the whole region has left many valley floors filled with large natural lakes, and over the last two decades construction of major hydroelectric and flood control projects has created additional large lakes in some of the valleys. On the Columbia River the Mica Project forms Kinbasket Lake Reservoir, a large lake in the Rocky Mountain Trench extending 200 km from Donald to Valemount in the northwest corner of the basin. The Revelstoke Project, presently under construction, will control water levels in the 130 km section of the Columbia River downstream of Mica Dam. The Arrow Lakes Reservoir forms a 233 km long lake from the City of Revelstoke downstream to the Hugh Keenleyside Dam just north of the City of Castlegar.

The completion of the Libby Dam Project on the Kootenai River in Montana created the 130 km long Koocanusa Lake Reservoir, extending north through the Rocky Mountain Trench to Wardner, B.C.. Storage behind

Libby Dam, and the Duncan Dam project on the Duncan River at the north end of Kootenay Lake, produce well regulated flows in the Kootenay Lake region.

The Murphy Creek damsite (Fig. 1-2) is on the Columbia River about 3 km upstream of Trail, B.C. and 23 km downstream of the confluence of the Columbia and Kootenay rivers. The total drainage area above the site is 88 000 km^2, of which the Kootenay River drains 50 000 km^2, or 57 percent.

Access to the right abutment of the Murphy Creek Project site is by Provincial Highway 22 while access to the left abutment is by unimproved road from Trail. Castlegar, 16 km north of the site via Highway 22, is located on southern trans-provincial Highway 3 approximately half way between Vancouver and Calgary and is the focal point of three major highways serving the West Kootenays. Daily scheduled commercial air service is available to Castlegar. A line of the CPR passes the site on the right abutment.

2.2 DEVELOPMENT OF THE COLUMBIA RIVER

A profile of the Columbia and Kootenay rivers is shown on Fig. 2-1.

Existing storage projects on the Columbia River system upstream of the Murphy Creek Project provide over 31.3 x 10^{9}m^3 of live storage. These include Mica Dam (14.8 x 10^{9}m^3) and Hugh Keenleyside Dam (8.8 x 10^{9}m^3) on the Columbia River, Libby Dam (6.0 x 10^{9}m^3) on the Kootenai River and Duncan Dam (1.7 x 10^{9}m^3) on the Duncan River, a tributary of Kootenay Lake. Kootenay Lake is regulated to a small degree by the Corra Linn Project (Fig. 1-1). All other hydroelectric projects above the Murphy Creek site are run-of-river projects with no significant live storage. The Revelstoke Project, presently under construction, will also be operated primarily as a run-of-river project.

Operation of 19.1 x $10^9 m^3$ of storage at Mica, Hugh Keenleyside and Duncan projects is governed by Assured Operating Plans (AOP) prepared each year for the sixth succeeding year in accordance with Annex A of the Columbia River Treaty.

The Keenleyside Powerplant and Murphy Creek projects would utilize the 31 m of available head between the Arrow Lakes Reservoir and the Murphy Creek damsite.

Based on B.C. Hydro's 1983 Electric System Plan[10] the first unit of the proposed Keenleyside Powerplant is scheduled to be in service in August 1992. The first unit of the proposed Murphy Creek Project is scheduled to be in service in July 1993. However, a change in the system plan in the next few years could cause an advancement or delay in these dates.

2.3 REGIONAL GEOLOGY

(a) Regional Setting

The Murphy Creek Project is located within that part of the Columbia River valley which marks the boundary between the Rossland Range of the Monashee Mountains, and the Bonnington and Norns Ranges of the Selkirk Mountains. The ranges all contain peaks above El. 3000 m.

The geology of the Columbia River Valley in the region of the proposed Murphy Creek Project is typical of the glaciated valleys of southern British Columbia. Bedrock, glacial drift and recent alluvial deposits dominate the valley.

(b) Bedrock Geology

The area is underlain mostly by plutonic rocks of the Nelson batholith, which consist principally of granite and granodiorite. The age of these rocks is Upper Jurassic to Lower Cretaceous.

The remaining bedrock in the project area consists mostly of sedimentary and igneous rocks, both intrusive and extrusive, generally metamorphosed and ranging in age from Lower Paleozoic to Cenozoic (Tertiary).

The bedrock formations found in the project area are shown on Fig. 2-2.

The structure of the area is closely related to and influenced by the emplacement of the Nelson batholith. However, most of the major structures were formed before the emplacement of the Nelson batholith and its satellites. The detailed structure of the area is complex with contacts between formations frequently gradational and poorly defined. The rocks are highly matamorphosed, strongly foliated and jointed. The strike of the gneissosity is somewhat erratic although regionally it conforms to the boundaries of the Nelson batholith. Many of the Nelson plutonic rocks also exhibit a gneissic foliation, though locally the rocks vary from massive to moderately well foliated.

There are no known major faults near the site. The regional fault system closest to the project site is the Fraser-Pasayten Fault, about 200 km to the southwest. The thrust faults of the Rocky Mountains are about 150 to 300 km to the northeast. Most of the other known faults in the region occur to the south and southeast, outside the Nelson batholith in older sedimentary and volcanic rocks.

(c) Surficial Geology

The most prominent unconsolidated deposits in the Columbia valley are remnants of terraces containing fluvial and glaciofluvial deposits consisting mainly of sand, gravel, cobbles and boulders and minor amounts of silt. Glaciers retreating from the Columbia River valley provided large volumes of glacial outwash that was deposited in the river bottoms. Deposition in the Columbia River valley was supplemented by sands and gravels deposited at the mouths of tributaries which may have carried substantial flows of meltwater. In post-glacial time the Columbia and Kootenay rivers eroded and reworked the mass of stratified drift to form a series of river terraces. Hence, the terraces on both sides of the river are remnants of a continuous fill of sediment through which the Columbia River has cut down about 150 m to its present level.

Upstream of the proposed Murphy Creek damsite, three terrace levels (identified as Els. 579 m, 518 m and 458 m) are prominent. The El. 579 m terrace exists mainly upstream and around Castlegar. The terrace containing the buried channel of the Kootenay River between Brilliant Powerplant and the Castlegar airport is about El. 579 m. Some of the El. 518 m terrace lies along the reach of the Columbia River valley south of Castlegar. The El. 458 m terrace is more pronounced downstream of Castlegar. South Castlegar and the communities of Blueberry Creek and Genelle are located on this terrace. In general, the deposits upstream of Castlegar are finer grained than those found downstream and traces of clay have been encountered in low-lying areas of Pass Creek and North Castlegar.

The valley bottom of the Columbia River near the Murphy Creek site has some large islands and bars formed by boulders and gravel with small amounts of sand. In the low-lying flat areas approximately 5 to 10 m above the present river level the materials are generally fine silty sands, probably overbank deposits (deposits laid down during flood conditions).

2.4 CLIMATE

Precipitation in the Columbia River basin develops mainly from the flow of moist low pressure weather systems which move eastward through the region from the Pacific Ocean.[11] The average annual precipitation in the basin (about 1000 mm) is more moderate than for basins of the Coast Range, which are exposed to the full impact of the Pacific weather systems.

As the basin topography exerts a strong influence on local climate, marked contrasts are observed between precipitation amounts at different locations in the basin. For example, the average annual precipitation is 370 mm at Canal Flats in the Rocky Mountain Trench, and is 2170 mm at Mount Fidelity, a high elevation station.

The summer climate is usually very hot and dry with average daily maxima and minima temperatures of about 25°C and 15°C respectively. In the winter the average daily maxima and minima temperatures are about 0°C and -5°C respectively. The presence of several large reservoirs in the basin has a slight moderating effect on temperatures at locations adjacent to the reservoir. In general, temperatures throughout the basin tend to be more uniform than precipitation.

SECTION 3.0 - PROJECT DESCRIPTION

PROPOSED PROJECT

During Preliminary Design studies the Murphy Creek Project was combined with the Keenleyside Powerplant Project and the Keenleyside-Murphy Transmission to form the Keenleyside-Murphy Project.

The main Murphy Creek project structures would be located, at their closest points, approximately 250 m from the community of Oasis and approximately 500 m from the community of Rivervale. The existing highway and railway between Oasis and the project site would be relocated (Fig. 3-1).

The proposed arrangement of structures (Fig. 3-1) includes an earth/rockfill dam with till core in the river channel. The powerhouse would be located on the right bank and would house five vertical shaft Kaplan units. A five-bay spillway would be located between the powerhouse and the dam.

The Murphy Creek reservoir would have limited storage and the project would be designed and operated with average monthly outflows approximately equal to inflows as required to meet Columbia River Treaty requirements and load-factoring at the generating plant. The project would have a rated head of 13 m and an installed generating capacity of 300 MW.

The proposed project is described in detail in Section 9.0.

The project arrangement described above is based on the hydrologic, power, hydraulic and geologic studies described in the following sections.

SECTION 4.0 - HYDROLOGIC AND POWER STUDIES

4.1 GENERAL

Hydrologic studies for the Murphy Creek Project comprised mainly an assessment of the probable maximum flood and flood frequencies to determine the capacity of the spillway and the river diversion system during construction. Estimates were made of the Probable Maximum Flood,[12] Spillway Design Flood[13] and Diversion Design Flood.[13]

The basic data on flows, reservoir levels, tailwater levels and heads as used in the power studies were provided by the System Engineering Division. Studies were performed on a computer model developed in 1979 specifically to simulate operation of the Keenleyside Powerplant, the Murphy Creek Project and Cominco's Brilliant plant on the Kootenay River. The studies were carried out for several different operating regimes for the Columbia River storage, the two most significant being as follows:

Regime 1

This operating regime is based on the Columbia River Treaty - Assured Operating Plan (AOP) developed for the operating year 1985/86. It is assumed that development of the Keenleyside-Murphy Project would not significantly alter the existing operation of the Columbia River System which is considered to be fairly representative of the first few years of operation at the Murphy Creek Project and Keenleyside Powerplant.

Regime 2

With an expected life of at least 70 years, the Murphy Creek Project and Keenleyside Powerplant would be operated well beyond year 2024, the earliest possible expiration date for the Columbia River Treaty. Regime 2 was developed to better reflect more flexible, future operating conditions that would maximize generation at Keenleyside and Murphy and would mitigate some of the environmental impacts caused by reservoir drawdown.

For most of the project life Regime 2 is expected to be much more representative of operation than Regime 1. For this reason Regime 2 has been chosen as the basis for project design. When the operating flexibility of the Columbia River Treaty projects improves following expiration of the Treaty, it is expected that the operating regime may be further refined to increase power generation, improve flood control at Trail and reduce the environmental impacts of reservoir drawdown.

For both Regime 1 and Regime 2, drawdown of reservoir water levels at the dam for generation of electrical energy would be limited to 2.0 m during March, April and May of each year and 1.5 m in other months. Drawdown upstream of the dam would be slightly less, depending on location and inflow. Daily fluctuations of tailwater level at the dam for operational purposes would be limited to 3.0 m, with an hourly variation limited to 1.0 m. Studies to optimize the amount of Murphy Creek Reservoir drawdown for generation will be conducted during Phase 2.

4.2 PROBABLE MAXIMUM FLOOD

The Probable Maximum Flood (PMF) study[12] for the Murphy Creek Project was carried out as part of the Feasibility and Preliminary Design (Phase 1) Studies. The PMF represents the flood characteristics that are considered to be the most severe "reasonably possible" at the project location.

A mathematical model, the UBC Watershed Model, was used to simulate streamflow in the Columbia River basin. The basin was divided into 12 sub-basins, and the model was calibrated for each of the sub-basins, using hydrometeorological data for the years 1970-77. This period includes several high-runoff and several low-runoff years. A second mathematical model, the UBC Flow Model, was used to route and combine runoff from each of the 12 sub-basins, incorporate reservoir operations at the Treaty projects (Mica, Keenleyside, Libby, Duncan) and to simulate hydrographs at the Murphy Creek Project site.

In simulating streamflow at the Murphy Creek Project site by the two models, the most important natural factors were recognized to be the snow quantity and its spatial distribution, the sequence of spring and summer temperatures, and the spring and summer rainfall. The PMF was determined by evaluating the natural factors and reservoir operations which would influence the peak flow at the Murphy Creek Project, and by combining these at their maximum values and most critical times.

The analysis indicated that the PMF would result from the occurrence of the Probable Maximum Rain (PMR) event at a critical time following the peak of the probable maximum snowmelt runoff. The snowmelt prior to the PMR event would be produced by the joint occurrence of extremely high winter snowpacks and extremely high spring temperatures. The conditions that would maximize the aggregate snowmelt production and the volume and distribution of the probable maximum snowmelt flood would result from the most severe combination of a 1000-year return period snowpack accumulation and a 1000-year return period temperature sequence. The temperature sequence was assumed to intensify over a period of three weeks and culminate in temperatures that would produce maximum snowmelt rates over the snow covered area.

The characteristics of the PMR storm were determined by using standard hydrometeorological procedures for maximizing the parameters of the most severe storm that has occurred within the general vicinity of the project drainage area. The PMR event would be greater in scale and intensity than any historical storm and would be transposed and centered on the Arrow Lakes Reservoir drainage basin over a 3-day period. The PMR would provide a variable amount of rainfall from over 180 mm in the center of the storm to lesser amounts on adjacent areas in the Columbia River basin.

Because of the effect of multiple reservoir developments upstream of the Murphy Creek Project and the resulting modifications to the flood hydrographs in the basin, the runoff conditions leading to maximum discharges under natural and regulated conditions would not be the same. The Treaty reservoirs, operated according to the Columbia River Treaty Flood Control Operating Plan, provide relatively effective flood control for the Murphy Creek Project up until 31 July, the date by which filling of the reservoirs would be completed. The major flood control benefit was found to occur early in the flood period when the Treaty reservoirs are filling and regulated flows at the Murphy Creek Project site are substantially below unregulated flows. However, once the reservoirs reach their normal maximum levels, regulated flows were shown to be approximately equal to unregulated flows since additional surcharge above normal maximum levels would not normally be permitted.

In summary, runoff conditions resulting in the PMF at the Murphy Creek Project would be produced by an extremely high winter snowpack, a wet and cool weather sequence from April to early July, an extremely high temperature sequence from then until the end of July, and the subsequent occurrence of the PMR event at the time when the flood control capability in the Treaty reservoirs would not be available to attenuate flows. Under these conditions, the PMF peak of 13,100 m^3/s would be produced in early August at the Murphy Creek Project. Hydrographs for PMF conditions at the Murphy Creek Project are shown on Fig. 4-1.

A recommendation of the PMF study was that consideration be given to postponing the Mica reservoir filling date from 31 July to the end of August during those years with a high runoff potential. Such an extension of the refill period for extremely large floods could reduce the PMF for the project by as much as 15 percent.

4.3 REGULATED FLOOD FREQUENCY CURVE

The Regulated Flood Frequency Curve studies[13] were carried out to derive an estimate of the Spillway Design Flood (SDF) and the Diversion Design Flood (DDF). The SDF was defined as the 200-year return period flood and future upstream reservoir elevations at Castlegar during this flood would be maintained close to the natural elevation. The term SDF has been used in a number of previous studies to describe the 200-year flood. The DDF was tentatively selected as the 30-year return period flood.

A key element of the study was the determination of how operational constraints could lead to storage refill of upstream Treaty projects prior to the normal 31 July target date. Constraints considered in this study were the 95 percent confidence-of-refill requirement for Columbia River Treaty storage, possible outflow restrictions at the Mica Project during the refill period, the effect of errors in forecasting seasonal runoff, and flood control objectives along the main stem of the Columbia River.

It was assumed in the study that upstream Treaty dams were operational in each year of the 1913-79 discharge record for the Columbia River at Birchbank/Trail. Given a set of operational assumptions, projections of the refill date for Treaty storage were made for each year. The natural flows at the Murphy Creek site were examined to determine the highest flows after the projected date of storage refill for each year. A frequency analysis was made using the log-normal distribution of these annual peak regulated flows, and from this a regulated flood frequency curve was derived. Several regulated flood frequency curves were derived, each assuming different modes of Treaty storage operation.

The flood risk at the Murphy Creek Project was shown to be sensitive to the date of refill of storage, which depends on whether Treaty storage at the Mica Project would be used in May-June for flood control in the U.S.A. or in July for flood control in Canada. The regulated flood frequency curve was shown to be a composite relationship and was developed from constituent curves that separately represent the project flood risk under various conditions of basin wide flood risk. The circumstances under which the different assumptions for Canadian Treaty storage operation would apply would depend upon the flood risk in a given year. For some years, the constraints of flood control in the U.S.A. imposed by major basin wide flood risk would be applicable. For the more frequent low-risk years, however, flood control interests would be automatically accommodated through the implementation of Assured Operating Plans. For a limited return period range, the project regulated flood frequency curve must be represented by a transition between the two regulated flood frequency curves developed for low and high basin wide flood risk conditions. Furthermore, every effort would be made to control floods to less than the existing 7930 m^3/s major flood control objective at Trail.

The study also examined the validity of the regulated flood frequency curve through flood simulation analysis, which investigated limitations that could arise from the selection of the 1913-79 period of record for flood frequency analysis. The regulated flood peaks obtained in the simulation analyses were shown to be compatible with the regulated flood frequency curve for the project.

The Murphy Creek Project flood frequency curve is shown on Fig. 4-2. From this curve, the 200-year return period flow was 7590 m^3/s, and the DDF was tentatively selected as the 30-year return period flow, 6310 m^3/s.

The Regulated Flood Frequency study concluded that a modified July operation at upstream Treaty projects, achieving storage refill no earlier than 31 July, could result in a reduction of up to 15 percent in the design floods for the Murphy Creek Project. However, such a modified operation must be shown to provide the same level of flood control effectiveness at The Dalles Project in the U.S.A. as is provided by the existing flood control plan.

4.4 POWER STUDIES - REGIME 1

The system power studies were performed using a monthly time interval in the simulation. Regulated reservoir levels and releases for both the Columbia and Kootenay rivers were based on a 35-year period of historical stream flows from August 1940 to July 1975 with regulation in accordance with the 1985/86 Assured Operating Plan.

Operating Regime 1 is essentially the existing regime for the Arrow Lakes Reservoir (behind Keenleyside Dam). Regime 1 would use, in most years, close to the maximum available live storage which would result in a fairly long period, generally mid-January to mid-June, when the Arrow Lakes Reservoir would be partially evacuated. In this situation, the Murphy Creek Reservoir level would also be lowered to provide the required discharge capability at Keenleyside. For the balance of the year the Murphy Creek Reservoir level would be maintained as near as possible to the normal operating level of El. 424 m, as measured at the Castlegar Ferry water level gauge. This level would be subject to daily fluctuations up to 2.0 m, required for operational purposes (Section 4.1).

A summary of simulated monthly reservoir and tailwater levels at the Murphy Creek Project, and monthly discharge hydrographs, based on Regime 1 is shown on Fig. 4-3. A summary of the reservoir level profiles is shown on Fig. 4-4.

The 35-year mean monthly total releases from Murphy would range from about 1350 m^3/s to 3400 m^3/s with the highest releases generally in January and February during Arrow Lakes reservoir drawdown. The 35-year mean discharge from Murphy based on this study is 2050 m^3/s.

Using statistical data on heads and discharges produced from the SED simulations, power studies were conducted to optimize installed capacity and compare alternative types of generating units (Section 7.0). For Regime 1 the optimum installed capacity was found to be 260 MW. The power study results for this regime are as follows:

Installed Capacity	260 MW
Average Annual Energy Total	1400 GW.h*
Average Annual Spill	580 m^3/s (28%)
Average Annual Plant Downtime due to Insufficient Head	6%**

Note: * Losses of 120 GW.h at Brilliant power station have been deducted from the average annual energy at Murphy.

** Minimum net head for operation, 6.4 m.

4.5 POWER STUDIES - REGIME 2

Regime 2 would involve a reduction of about $2.5 \times 10^9 m^3$ in the mean annual drawdown volume of the Arrow Lakes Reservoir. Under this regime the Murphy Creek Reservoir would rarely have to be lowered to increase Keenleyside discharges capacity; hence the reservoir would generally be at the normal operating level of El. 424 m. This level would be subject to daily fluctuations up to 2.0 m, required for operational purposes (Section 4.1).

A summary of simulated monthly reservoir and tailwater levels at the Murphy Creek Project, and monthly discharge hydrographs, based on Regime 2 is shown on Fig. 4-5. A summary of reservoir level profiles is shown on Fig. 4-6.

For the 35-year simulation with Regime 2, mean downtime at Keenleyside Powerplant[9] would be about 2.6 months per year. The operating period for the Keenleyside Powerplant could be extended by drawing down the Murphy reservoir as the Arrow Lake Reservoir is lowered; thereby maintaining the minimum gross head as required for the turbines. However, because of the backwater effects in the very small Murphy Creek reservoir, the extent to which this could be done would be limited, after which the losses at Murphy Creek would exceed the gains at Keenleyside and Brilliant. In the present studies it has been assumed that 1.0 m of such drawdown of the tailwater level at Keenleyside would be utilized, reducing the mean annual downtime to about 2.4 months per year. Studies to optimize the amount of Murphy drawdown will be conducted during the next phase of studies.

Using statistical data on heads and discharges produced from the simulations, power studies were conducted to optimize installed capacities. For Regime 2 the optimum installed capacity was found to be 300 MW. The power study results for this case are as follows:

Installed Capacity	300 MW
Average Annual Energy, Total	1970 GW.h*
Average Annual Spill	100 m^3/s

Note: * Losses of 145 GW.h at Brilliant power station have been deducted from the average annual energy at Murphy.

Based on an average annual discharge at the Murphy Creek Project site available for power generation of 2050 m^3/s, the proposed installed plant hydraulic capacity of 2620 m^3/s would utilize 96 percent of the flow available in the river under Regime 2. The head, discharge and monthly average power potential under Regime 2 are shown graphically on Fig. 4-7.

4.6 SEDIMENT REGIME AND RIVER MORPHOLOGY

The Columbia River channel upstream from the Murphy Creek damsite and downstream from the Keenleyside Dam is composed primarily of gravelly, moderately coarse to very coarse textured glacio-fluvial and ice-contact deposits.[11] Along this reach, there are 17 identifiable creeks which flow into the main river. Near the mouths of these creeks, the main-stem channel bed is composed partly of the alluvial material transported by these creeks.

Between Champion Creek and the damsite, the left bank of the Columbia River is made up of some loose colluvium (erosion material) as well as glacio-fluvial deposits. The right bank has less colluvial material. Gravel bars exist at low water levels in the Columbia River channel between China Creek and Birchbank. In this reach, the channel is wider and has a flatter slope than that of the channel in adjacent reaches.

Measurements of suspended sediment concentration in the Columbia River at Birchbank have been taken since August 1965. The pattern of monthly suspended sediment loads was altered in 1968 with the construction of the Keenleyside Dam, and the periods of record before and after 1968 were too short to warrant any quantitative comparison. However, a perusal of the suspended sediment load record indicated that the May-to-September suspended sediment loads decreased after the construction of Keenleyside Dam and since regulation of flows according to the Columbia Treaty.

Suspended sediment samples were obtained in June 1980 from nine of the creeks that discharge into this reach of the Columbia River. Very little suspended sediment was carried by these creeks; concentrations determined from the samples ranged from 0 mg/ℓ to 2.5 mg/ℓ. No bed-load measurements were made on the Columbia River or on any of the tributary creeks.

Velocities in the Murphy Creek Project reservoir would be at least as high as those in the reservoirs behind the Brilliant and Keenleyside dams. Almost all of the small amounts of suspended sediment passing through these upstream reservoirs would be discharged from the Murphy Creek reservoir.

The upstream dams effectively trap all bed-material within their reservoirs, and thereby halt any movement of bed-load beyond these dams. Therefore, almost all of the sediment which might accumulate in the reservoir would originate from bed-load movement in the 17 tributary creeks.

As only three of these tributary creeks had been gauged for a significant period of time, it was necessary to develop estimates of peak flows in the tributary creeks. Peak flow hydrographs were developed for ungauged creeks. Bed-load transport rates were then estimated using Shields' formula. Factors which lead to uncertainty in the estimated bed-load transport rates were considered by making assumptions which would tend to over-estimate the transport rates.

Some bank sloughing would be expected when the level of the Columbia River is raised (Section 8.0). An estimate of the maximum potential amount of erodible shoreline material which could be deposited in the reservoir is $6 \times 10^6 m^3$. The major portion of any deposited material would remain in the reservoir with only small portions of fine-grained material flushed out. Additional studies on this subject will be made in Phase 2.

The Phase 1 studies led to the following conclusions:

1. The river channel is morphologically stable at present.

2. Sediment is carried into the Columbia River by the tributary creeks as bed-load only in times of flood.

3. Higher water levels in the reservoir after completion of the Murphy Creek Dam would result in the formation of new creek deltas.

4. The useful life of the project reservoir, defined as the time taken for sediment to occupy the dead storage volume and half of the live storage volume, would be in excess of 3000 years.

4.7 GROUNDWATER

(a) Present Conditions

Groundwater studies[14] indicated that, in the Murphy Creek Project area, most of the groundwater originates as precipitation. With the exception of a few areas, the Columbia River is not a source of adjacent groundwater. Groundwater from upland areas moves through aquifers towards the river. The water level in the river affects the water table in the aquifers adjacent to the river, but, except for the few aquifers which are fed by the river, has no influence on the quantity and quality of groundwater flow. The slope of the water table toward the river is determined by the rate of groundwater flow and the aquifer permeability and is generally very flat.

Most groundwater recharge in the area is intermittent because of the local climate. Significant recharge takes place only during the period of high run-off in early spring. In spite of the fact that recharge is seasonal, groundwater flow is reasonably steady because of the long flow paths, and because the flow is continuous except in small local flow systems.

In the project area, there are three main types of aquifers supplying water to wells:

1. Fractured Rock - Wells drilled into this type of aquifer have low capacity and are often used for domestic supply. Water from this type of well is usually highly mineralized. There are a number of these wells in the Oasis area.

2. Alluvial Deposit - this type of deposit exists at the mouths of tributary creeks. Wells drilled in alluvial fans may have moderate capacity, depending on their design and construction.

3. Ice-contact sediment - Wells in this type of aquifer may have moderate capacity if they are properly constructed and are completed in permeable parts of the deposits. Since ice-contact sediment contains a non-homogenous mixture of materials with high and low permeabilities, wells in the same aquifer may have very different capacities and water quality. Most of the wells in the area are in this type of aquifer.

The aquifer underlying the Selkirk College peninsula is surrounded by the Kootenay and Columbia Rivers on three sides. This aquifer probably contains water moving from the Kootenay River to the Columbia River, which is at a slightly lower elevation. The groundwater at Selkirk College is much less mineralized than that found elsewhere in the project area and is similar in quality to that in the Kootenay River.

(b) Post-Project Conditions

The major impact of the Murphy Creek Project would be to raise groundwater levels in the immediate vicinity of the reservoir. Wells which are not inundated should have higher water levels and, therefore, slightly higher capacities and lower pumping costs than before. Without mitigation measures, flooding of some basements could occur periodically, and the operation of several on-site sewage facilities could be affected.

In 1981, 39 piezometer holes were drilled in the areas of Castlegar, Blueberry Creek, Robson, Raspberry, Brilliant, Ootischenia, and Selkirk College. The water levels in these piezometers have been measured on a monthly basis and this data has been used to calibrate a groundwater computer model for the project area. Twenty-two additional piezometers were installed in early 1983 (Section 6.4). As additional groundwater data becomes available, the calibration of the computer model will be improved to predict groundwater levels in the project area more accurately.

Operation of the project would produce fluctuations in the water table in the immediate vicinity of the project which are greater in magnitude and more frequent in occurence than those which occur presently. However, it is considered that neither this factor, nor the raising of the water table, would have any effect on aquifer water quality.

After project construction, a higher proportion of recharge of the Selkirk College aquifer would originate in the Columbia River. However, it is considered that the quality of water in the aquifer would not be significantly affected.

Finally, there is a remote possibility that blasting activities at the Murphy Creek site would affect the rock structure in the aquifer underlying Oasis. This could possibly affect the water quality and capacity of wells in Oasis.

SECTION 5.0 - HYDRAULIC STUDIES

5.1 SPILLWAY DESIGN

The initial criterion for spillway design when higher reservoir levels were being considered was that during the routing of the flood with a recurrence interval of 200 years, no surcharge above the normal operating headpond elevation at Castlegar should result. Complete elimination of the surcharge is not possible for the normal operating headpond of El. 424 m, as the natural river elevation at Castlegar with the river discharging the 200-year flood is El. 425.6 m. In addition, as any practically-sized spillway located downstream would be less efficient than the natural river section, some backwater effect would always be present. The spillway facilities and drawdown at Murphy have therefore been designed to keep future water elevations at Castlegar close to the natural river elevation of 425.6 m for the 200-year flood.

The spillway would have a low ogee cross section and would consist of five bays of 16 m width as shown on Fig. 5-1. The spillway crest, at El. 404 m, would be submerged by the tailwater at all times.

An optimization study was undertaken to maximize the spillway discharge capacity by varying the crest elevation and length of the ogee section and the upstream and downstream apron elevations. It was found that due to the high level of submergence the discharge was not sensitive to small variations in these parameters. Fig. 5-2 shows the results of the study with the reservoir drawdown to El. 418.2 m at the dam, when discharging the 200-year flood of 7590 m^3/s, resulting in a water level at Castlegar of El. 425.8 m. The corresponding figures for the PMF are El. 424.2 m at the dam and El. 430.8 m at Castlegar. The upstream face of the ogee is at a 45° slope in order to increase the mass of the ogee section for structural stability and to provide greater resistance against uplift and vibration.

There is some possibility that the highly submerged flow over the spillway may cause vibration and pressure fluctuations under gate controlled operation, and this condition is being examined further. If considered necessary, a hydraulic spillway model would be constructed for studies during Phase 2 of preliminary design. Raising the ogee crest 3.0 m to El. 407 m would avoid a submerged condition and tend to stabilize the flow. However, additional spillway bays would then be required in order to retain the required discharge capacity, thus increasing the width and cost of the structure.

Flap gates are being considered to replace the top portion of the proposed radial gates. They would offer the advantage of more accurate reservoir level control because they would be unaffected by tailwater submergence and would also allow debris to be passed over the gates. This will be considered further as part of spillway optimization studies during Phase 2 of Preliminary Design.

In Sections 4.2 and 4.3 reference was made to the effects of a 1 month postponement of the filling date of the Mica Reservoir on the flood inflow peaks for Murphy. A reduction of 15 percent in peak values was considered possible and this could result in a reduction in spillway crest length.

5.2 BACKWATER STUDIES

As flooding at Castlegar is a primary consideration in establishing acceptable reservoir levels, studies of the backwater (slope of the water) from the dam up to Castlegar were made. This determines the water level at the dam that would result in a satisfactory level at Castlegar for any given river discharge.

Water elevation versus discharge relationships have been established for the full range of flows on the Columbia and Kootenay rivers and for 30 locations between Murphy and Keenleyside dams. Representative profiles are shown on Fig. 5-2. The backwater computer program HEC-2 was calibrated using four Water Survey of Canada gauging stations, 31 surveyed cross sections, and one surveyed profile. The program was then utilized to extrapolate to post-Murphy conditions and to higher floods. Due to the high quality of streamflow records it is expected that the water levels predicted by the program would be within a few tenths of a metre of the true levels. Other than a rating curve verification at the dam axis no further refinements to the backwater relationships are considered necessary.

The reservoir level of El. 424 m would be measured at the Castlegar ferry crossing and would be maintained at this level by controlling the water level at the dam until the spillway head requirements govern. This would occur at a flow of about 6000 m^3/s (Fig. 5-3). For higher flows the water level at Castlegar would begin to rise above El. 424 m. At a flow of 7590 m^3/s (200-year flood) the reservoir water level at Castlegar would be at about El. 425.8 m. This compares to the water level at Castlegar under natural river conditions of about El. 425.6 m for the same flow. Water levels at other locations on the reservoir would vary with discharge as shown on Fig. 5-2. The reservoir elevation at the dam during maximum powerplant operating conditions (discharge 2620 m^3/s), would be El. 423.3 m, and during operating conditions for mean flow (discharge 2050 m^3/s), would be El. 423.6 m.

A tailwater rating curve for the Brilliant Dam (Fig. 5-4) on the Kootenay River was derived using the same methodology as described above. The Brilliant tailwater elevation is a function of both the Columbia and the Kootenay flows. Considerable difficulty was experienced in calibration runs because of a hydraulic control 600 m downstream of Brilliant and a limited historical record of water levels. Despite this, a curve was developed to an accuracy of approximately 0.5 m at Brilliant. This data was used to assess the reduction in power generation at Brilliant due to the Murphy Creek reservoir.

5.3 CHANNEL IMPROVEMENT

Channel improvements to reduce head losses and increase power production were investigated at three locations: Tincup Rapids in Castlegar, Waneta Plaza about 3.5 km downstream from the Trail bridge and near Sunningdale about 1 km upstream from the Trail bridge.

Channel improvement through excavation at Tincup Rapids would reduce river elevations upstream in Castlegar and therefore allow a higher Murphy forebay for any given water elevation at Castlegar. However, with an estimated benefit/cost ratio of 0.15 the Tincup Rapids excavation was determined to be uneconomic.

An accurate estimate of the benefits from excavation at Waneta Plaza is presently not possible because few sections are available downstream from the Trail bridge. It was estimated that tailwater elevations at Murphy could be lowered by approximately 0.15 m at flows corresponding to the mean annual flow (2050 m^3/s). This represents about 1 percent of the rated head and will be considered further during Phase 2 of Preliminary Design studies.

Backwater studies carried out upstream of the Trail bridge indicate that if the rapids near Sunningdale were completely excavated, tailwater elevations at Murphy would be lowered by 0.64 m at river flows of 445 m^3/s, 0.18 m at 1370 m^3/s and zero at 3757 m^3/s. Based on the 35-year simulation of river flows, the net annual benefits from complete removal of the rapids would be about $320,000 (based on the March 1982 economic criteria).[15] A cost/benefit analysis will be carried out after hydrographic surveys are completed during Phase 2 Preliminary Design.

5.4 TRANSIENT STUDIES

The hydraulic transients due to the operation of the Murphy Creek Project were analysed using an implicit finite difference method. This method was found to be suitable for analysing natural channels and provided better control of time intervals. Surveyed river sections, as used in the backwater studies, were used to represent the river channel. Only preliminary results have been obtained from the study at this time. These indicate that the magnitude of surges resulting depend on the reservoir elevation and the amount and rate of change of flow.

Upstream of the Murphy Dam, a maximum upsurge of 1 m would occur at the dam following a complete load rejection at the station with the reservoir at the normal operating level of El. 423.6 m (discharge 2050 m^3/s). This surge height diminishes rapidly upstream and is almost completely dissipated before reaching Castlegar. The upsurge would be reduced to 0.8 m if, immediately following load rejection, two of the five spillway gates were opened at the normal rate to a partial opening of 1 m. Lower initial reservoir levels would give rise to higher upsurges. The maximum downsurge in the forebay on station start-up at a reservoir elevation of 423.6 m would be 0.9 m, assuming the five generating units were started up at 5-minute intervals. This downsurge would increase at lower reservoir elevations. A temporary loss of submergence at the intakes could be created therefore by start-up at lower reservoir elevations. This will be considered further during Phase 2 Preliminary Design.

Downstream of the Murphy Dam, the maximum upsurge would be 2.3 m on instantaneous station start up. The same rise in water surface would spread out over a period of 20 minutes for generating unit start-up at 5-minute intervals. The maximum immediate downsurge would be 2.1 m for a station load rejection event, reducing to 1.8 m when two spillway gates are partially opened to 1 m. Further study of these surges will be carried out in Phase 2, including the effect of rapid transfer of discharge from powerhouse to spillway and vice versa to maintain uniform river discharges.

A pumphouse, owned and operated by Cominco, is located approximately 2.8 km downstream of the dam. It supplies process and cooling water to the Cominco smelter in Trail as well as domestic water to Tadanac and Warfield. Back-up storage at the pumphouse provides flow for about half an hour. The minimum river flow for adequate pump submergence is 425 m^3/s. Studies were made to determine the effect of load rejection at this river flow on the pump submergence. It was found that if two of the spillway gates are automatically opened to 1 m following full load rejection, the downsurge at the pumphouse would be reduced to 0.06 m, which is considered acceptable for continued pump operation.

5.5 HYDRAULIC MODEL STUDIES

Hydraulic model studies are necessary to investigate some aspects of the project which cannot be formulated analytically. As described in Section 5.1 above, a spillway model would be used to examine the effects of high tailwater submergence. Possible adverse effects of the submergence could be pressure fluctuations, turbulence and wave generation, and an incorrectly assumed discharge coefficient. A comprehensive model is also necessary to confirm calculated diversion water levels as well as to carry out a full investigation of the various stages of the diversion scheme. Flow conditions at the power intakes, circulation patterns and the need for river training structures would also be studied. These models will be constructed in Phase 2 of Preliminary Design.

5.6 GAS SUPERSATURATION

Hydraulic calculations have shown that the jet issuing from beneath the spillway gates at Murphy would be submerged so that little or no air entrainment would take place. Therefore the proposed structures should not add to the dissolved gas present in the Columbia River during gated spill operations. This will be checked on the proposed comprehensive hydraulic model. In addition, dissolved gas prediction calculations have shown that, under normal free flow conditions such as would be necessary at high discharges, with the gates fully removed from the flow, there would be little or no increase in the dissolved gas concentration in the flow passing over the spillway.

SECTION 6.0 - SITE GEOLOGY AND INVESTIGATIONS

6.1 SITE INVESTIGATIONS

The geology in the vicinity of the Murphy Creek Project has been studied by various agencies. Bathymetric surveys were done by the Department of Public Works in 1946. Drilling was done by the Department of Northern Affairs and National Resources[16] in 1953 and 1955, and a geological report[17] was prepared by Dr. V. Dolmage in 1957.

Seismic refraction surveys, gravity surveys, geologic mapping, and drilling investigations[18] were carried out in 1979 and 1980. In 1979, four holes were diamond drilled along section AA on Fig. 6-1 which was the axis selected during the pre-feasibility studies.[2] Bedrock permeability tests were performed in one drill hole. Two seismic refraction lines were surveyed on the left bank of the river in order to obtain bedrock profiles and overburden thickness.

In 1980, investigations[18] concentrated on the overburden materials on the left abutment at the dam axis. Nine air rotary holes were drilled to bedrock and falling head permeability tests were performed in the overburden of two holes. In addition, gradation tests were performed on overburden samples to estimate permeability. Two additional seismic refraction lines were surveyed on the left abutment. A gravity survey of the damsite area was conducted to assist in defining the location of the left bank buried channel.

In 1981, geotechnical investigations[19] were concentrated in the vicinity of the left abutment and the river channel, and included overburden and bedrock foundation drilling, installation and monitoring of piezometers, overburden and bedrock permeability testing and the development of a pump test well. Eleven overburden holes were drilled in the vicinity of the left abutment and Billy Creek (Fig. 6-1). Standpipe piezometers were installed in seven of the drill holes and one drill hole was developed as a pump test well. Seven bedrock holes were drilled, with six of the holes located in the Columbia River, and one hole located on the left bank of the river near the waters edge. Two trenches were cut into the steep side slopes of the lower terrace on the left bank, and the exposures mapped in detail. A 96-hour pump test was performed in order to determine in-situ permeability of the granular material at depth in the left bank. The 1981 investigations provided information to optimize the location of the dam axis, to determine the permeability of material at depth on the left bank, to determine geotechnical properties for the design of a seepage cut-off and to assess potential grouting requirements in the river bed.

In 1982, investigations[20] were concentrated in the vicinity of the left abutment, and included overburden drilling to bedrock (no rock or soil cores were taken) and installation and monitoring of piezometers. Four air rotary holes were drilled along Axis I on the left abutment in order to refine the bedrock contours. On the right bank, test pits were dug in areas between bedrock outcrops in order to determine depth to bedrock.

6.2 SITE GEOLOGY

(a) General

At the Murphy Creek Project site the Columbia valley is approximately 1300 m wide and bounded on both sides by steep rock walls. The present Columbia River occupies the western side of the valley and is bounded by bedrock towards the west and gravel terraces to the east. The gravel terraces forming the eastern (left) bank of the river extend back 1000 m before encountering bedrock. Rock islands and rock ridges extend into the river for about 30 m from the west (right) bank before the rock dips under river gravels. A series of bedrock ledges rise gradually from the west riverbank to the valley slope located approximately 300 m from the river.

(b) Bedrock Conditions

The bedrock exposed at the site (Fig. 6-2) is a medium to fine-grained, light to dark grey granodiorite cut by andesite dykes. All rocks show evidence of low grade metamorphism but are highly competent and durable.

The granodiorite displays a gneissic foliation dipping to the northwest at about 40°. Fine-grained, dark grey andesite dykes ranging in thickness from 0.5 to 35 m and dipping eastwards at about 45° were emplaced along parallel planes of weakness in the granodiorite. The contacts with the granitic rocks are sharp and tight.

Two joint sets are distinguishable: the most continuous set dips easterly at 45° and is spaced 1 to 2 m. A less continuous set is near vertical and strikes east-west. Joints are mostly tight and, in many cases, are calcite infilled. In general, the quality of the foundation bedrock is good with very high rock strengths.

No faults were mapped on the surface or intersected in drilling. A 10 cm shear zone was encountered in DH81-M66,[19] located in the river channel just upstream of Axis I (Fig. 6-1), at about El. 372.6 m. Hole DH79-M40 (Figs. 6-1 and 6-3, Section A-A), drilled on the right abutment intersected a closely jointed zone that, when projected to the surface, corresponds to a sand filled gully. The outcrop pattern on the right abutment is the result of preferential glacial erosion of the most jointed rocks.

Drill holes DH81-M67 and M69[19] (Figs. 6-1 and 6-3, Sections B-B and A-A respectively), located in the river channel indicated an apparent 0.4 m to 0.5 m thick sand and gravel infilled fracture in the granodiorite between Els. 390.5 m and 391.2 m. The orientation of the apparent fracture has not been established.

The interpreted bedrock contours (Figs. 6-1, 6-2) indicate a potential buried river channel in the left bank parallel to the Columbia River. The left bank deposits may be as deep as 130 m along this section. Bedrock contours in the Columbia River channel west and downstream of Mary's Rock indicate the presence of shallow bedrock at about El. 400 m. However, as this is based on gravity survey data and riverbed contours have not yet been defined in this area by drilling due to swift river currents, they are not considered reliable.

In the riverbed bedrock outcrops at hole DH81-M69.[19] At other hole locations, river gravel overlies bedrock. Southward from DH81-M64 to M66, river gravel thickness ranges from about 32 m to 25 m, and from DH81-M67 to M69 the river gravel thickness ranges from about 4 m to 0 m. Drilling results[19] also show that bedrock drops off approximately 50 m upstream of Axis I. The bedrock types consist primarily of medium to coarse grained, hard granodiorite intruded by fine-grained, hard andesite dyke. Drill holes in the river indicate that the bedrock surface is weathered to a depth of less than 2 m.

Permeability tests show that the bedrock is generally tight.

(c) Overburden Conditions

On the right bank of the river overburden is confined to patches of sand infilling erosion gullies between outcrops. In 1982,[20] twelve test pits were dug in gully infill areas between bedrock outcrops

(Fig. 6-2) on the right bank both upstream and downstream of Axis II. Bedrock was encountered in eight test pits at depths ranging from 2.5 to 7.5 m below ground surface. Generally, the overburden is granular material ranging from silt and fine sand to mixtures of sand, gravel and cobbles with boulders up to 1 m diameter.

On the left bank the gravel banks rise from the river at a 30° slope to El. 455 m and form a gently sloping lower terrace rising to El. 470 m (Fig 6-3). This lower terrace ranges in width between 150 and 250 m and extends about 1500 m both upstream and downstream from the dam axis. A second steep gravel slope rises to an upper terrace that slopes up gently towards the east between El. 510 m and El. 560 m. This upper terrace extends upstream of the dam axis for 800 m and downstream for 1500 m and has a maximum width of 500 m. Both terraces are incised by a deep V-shaped gully created by Billy Creek.

The estimated average in-situ permeability of the channel infill deposits on the left bank ranges from 10^{-1} to 10^{-2} cm/s. Highly permeable zones may occur at depth and near the bedrock surface.

The groundwater level was found to increase during drilling in all four 1982 drill holes.[20] It was also observed that the level of the water table monitored in piezometers away from the river responded rapidly to fluctuating river levels.

From 19 November 1982 to 7 January 1983, three drill holes near the dam axis were instrumented with continously recording water level monitors. River stage data at the damsite were calculated from measurements made at Birchbank, 4 km upstream of the damsite. Several fluctuations of the ground water level of up to 2 m occurred during the period of record. River flows varied from

1130 m^3/s to 2400 m^3/s, corresponding to river stage fluctuations of about 1.5 m.

Analysis of the data was carried out[21] using a tidally influenced coastal aquifer model and a steady state pumping test model. The calculated overall transmissivity of the aquifer is in the order of 0.01 m^2/s which corresponds to a permeability of 2×10^{-2} cm/s for the assumed 50 m aquifer thickness.

River depths in the explored section of river ranged from 7.3 m to 12.2 m. The riverbed overburden is a mixture of sand, gravel and cobbles to a maximum depth of 32.2 m (at DH81-M64). Large boulders up to 2 m in diameter were encountered in drill holes DH81-M64, M65 and M66 which are all located upstream of Axis I.

6.3 GEOLOGICAL CONDITIONS AFFECTING DESIGN

The major geological conditions that could affect the design of the structures are as follows:

1. Investigations to date reveal the existence of a buried channel estimated to extend to about El. 335 m beneath the left bank. The infilling material has been shown to be very pervious with permeabilities of 10^{-1} to 10^{-2} cm/s. To ensure a safe dam and to reduce seepage losses a seepage cut-off would be required. Random boulders up to 2.5 m in diameter may complicate the construction of the seepage cut-off wall.

2. The site bedrock is of excellent quality and would provide a suitable foundation for the concrete structures. Bedrock permeability is low, generally less than 10^{-6} cm/s. However, permeability tests performed in three river channel holes indicated bedrock permeability ranging from 10^{-4} to 10^{-3} cm/s. This

indicates the upper rock may require grouting but it is anticipated that grouting requirements would not be extensive. No bedrock foundation problems are anticipated.

3. Gravity survey data indicate the presence of shallow bedrock (about El. 400 m) in the Columbia River channel west and downstream of Mary's Rock. Riverbed and bedrock contours have not been defined in this area by drilling due to swift river currents.

4. Two of the six drill holes (DH81-M67 and M69) located in the river channel indicated the presence of apparent 0.4 m to 0.5 m thick fracture zones with sand and gravel infilling in the granodiorite at about El. 390 m. The orientation of the apparent fractures has not been established.

6.4 SEISMIC ASSESSMENT

The 1970 Seismic Zone Map for Canada places the project area on the boundary of Seismic Zones 0 and 1, an area of very low to low earthquake hazard.

Seismic records for the area are available from 1899 to present and have been obtained from Energy, Mines and Resources Canada, Pacific Geoscience Centre (PGC). No earthquakes have been recorded within about 50 km of the site. The strongest earthquake recorded within 200 km of the site is a magnitude 5.5 event which occurred in 1942 about 150 km southwest of the project site.

Since 1899, the Keenleyside-Murphy area has experienced a maximum horizontal acceleration of 1 percent of gravity, (below the human perception level of about 1.5 percent), caused by strong earthquakes in the West Coast-Puget Sound area and by lower magnitude earthquakes closer to the sites. Using the Cornell risk estimation method, which is to be adopted for the 1985 National Building Code of Canada, PGC has calculated that a peak horizontal acceleration of 5 percent gravity and a peak horizontal velocity of 5.5 cm/s have an annual probability of exceedance of 1:475. [The exceedance level of 1:475 will be adopted in the 1985 National Building Code.] Theoretically, a magnitude 6 earthquake at a distance of 60 km could induce such an acceleration and velocity at the project site according to PGC attenuation relationships. However, there are no fault systems within a 60 km radius from the project site that would have the potential to generate a magnitude 6 earthquake.

The closest regional fault system is about 200 km to the southwest. There are numerous less extensive faults in the area, mostly to the south and east, but there has been little recorded seismic activity associated with these faults.

The Arrow Lakes reservoir behind Keenleyside Dam was filled in 1969. This did not cause any measurable increase in seismic activity.

Based on the nature of regional faulting and the recorded seismic history, a maximum credible earthquake of magnitude 6 has been assumed for the area, and a peak horizontal acceleration of 10 percent gravity (0.1g) has been adopted for design of major structures. The probability of having a magnitude 6 earthquake near the project site that would be capable of generating a peak horizontal acceleration of 10 percent gravity (0.1g) is extremely small from the point of view of geotectonic considerations as there are no active faults near the sites. However, this acceleration was used for the design of the existing major structures at Hugh Keenleyside Dam and is consistent with ICOLD [International Congress on Large Dams] recommendations.

The design of ancillary structures is to conform to the requirements of the 1985 National Building Code and an acceleration of 5 percent gravity (0.05g) has been adopted (probability of exceedance of 1:475).

6.5 RESERVOIR INVESTIGATIONS

In 1980, the proposed Murphy Creek reservoir shoreline was investigated. Terrain mapping based on air photo interpretation was carried out at a scale of 1:25,000 using the Environmental Land Use Committee (ELUC) Terrain Classification System.[22]

In 1981, an initial evaluation of the area geology and landforms was made.[23]

In the summer and fall of 1981, a reservoir shoreline assessment was carried out.[24] Thirty-nine boreholes totalling 1132 m were drilled adjacent to the proposed reservoir using a truck-mounted Becker hammer drill. Drill hole depths varied between 10 and 62 m. The soil was logged during the drilling and disturbed bag samples and undisturbed Shelby tube samples were collected for subsequent laboratory testing. Standard Penetration Tests (SPT) were also carried out at frequent intervals in each of the holes. A total of 535 disturbed bag samples and 10 undisturbed Shelby tube samples were collected during the field program. Standpipe piezometers were installed in each of the 39 drill holes. In addition to the drilling program, 216 slope exposures were mapped and six test pits dug. Samples were taken from both slope exposures and test pits for future lab testing. Survey sectioning was carried out at 18 locations considered to be representative of some of the critical slopes around the reservoir.

As part of the C.P. Rail Study[25] in 1981, 30 test pits were excavated and disturbed samples obtained and tested.

In 1983, a drilling program was carried out to provide additional information regarding groundwater and reservoir shoreline conditions. Twenty-two boreholes, totalling 420 m, were drilled adjacent to the proposed reservoir using a truck-mounted Becker hammer drill. Drill hole depths varied between 10 and 42 m. The soil was logged during drilling and disturbed bag samples were collected for subsequent lab testing. Standard Penetration Tests were carried out at various intervals in three drill holes. A total of 48 disturbed bag samples and 10 SPT split spoon samples were collected. Both the 1981 and 1983 piezometers are being read at regular intervals.

6.6 CONSTRUCTION MATERIAL INVESTIGATIONS

(a) General

The construction material (in place) requirements for the proposed project are:

Impervious material (glacial till)	22 000 m^3
Processed concrete aggregate	400 000 m^3
Granular fill	140 000 m^3
Rockfill (incl. riprap)	150 000 m^3
Shoreline protection fill	1 000 000 m^3

Some potential construction material sources were originally identified in 1979 from air photographs, topographic maps and general reconnaissance work.[3]

(b) Impervious Fill

During 1981, Areas 2, 3, 4 and 5 were investigated[19] as detailed on Table 6-1 (Fig. 6-4). A total of five drill holes (Area 2) and 28 test pits (Areas 2, 3, 4 and 5) were excavated. The auger drill rig was not able to penetrate a cobble and boulder cap overlying the till in Area 2. Following this program, Areas 2, 3 and 5 were rejected in favour of Area 4 as an impervious till source.

During 1982, Area 4 was investigated further by helicopter auger drilling (Fig. 6-5). Four drill holes totalling 67 m were completed by augering and coring. Laboratory testing[20] indicated the till would be suitable as an impervious material.

Adequate volumes in excess of the 22 000 m³ required are available from Area 4.

(c) Granular Fill and Concrete Aggregate

During 1980 and early 1981 Area 6 was investigated as a granular fill and concrete aggregate source.

During 1981, nine Becker drill holes totalling 192 m and 20 test pits (max. depth 7.3 m) were completed in Area 6. Bulk samples were collected and the laboratory test results[19] indicated the material to be satisfactory. However, due to site constraints and other factors such as possible alternative uses of the area for construction camp or recreational, Area 7 was investigated.

During 1981, two access road cut trenches were examined in Area 1 (contained within Area 7). During 1982, 38 Becker drill holes totalling 938 m and 17 test pits (max. depth 10 m) were completed in Area 7 (Fig. 6-6).

Laboratory tests[20] indicate the material in Area 7 to be suitable for both granular fill and concrete aggregate. Processing of the granular material would be required.

Approximately 1 million m³ of unprocessed granular material is available within Area 7.

(d) Rockfill

All the rockfill requirements would be met from the project rock excavations. The rock is of excellent quality.

(e) Shoreline Protection

Shoreline protection material would be obtained locally, where possible. Additional material would be available from Hugh Keenleyside project rock excavation, Murphy Creek project rock excavation or possibly from Area 7.

SECTION 7.0 - PROJECT ARRANGEMENT STUDIES

7.1 ALTERNATIVE PROJECT ARRANGEMENTS

(a) Background

During the prefeasibility[3] and feasibility[4] studies, four alternative project arrangements, F.1 to F.4, were compared (Fig. 7-1) and lead to the selection of Alternative F.3. This alternative would consist of a powerhouse located against the right abutment with the spillway in the centre, between the powerhouse and the earth-rockfill dam. The dam would be located in the river channel.

Based on the results of the prefeasibility and feasibility studies which showed relatively small differences in cost for the various alternatives, and the information available from early investigations for the preliminary design, several alternative project arrangements were considered and the more promising ones selected for comparative studies (Arrangements P.1, P.2 and P.3, Figs. 7-2, 7-3 and 7-4).

During the feasibility studies,[4] horizontal-shaft turbine units were tentatively selected. On the basis of further information available in the early stages of this Preliminary Design Study, vertical-shaft turbine units were also considered.

As part of the comparative studies, the powerhouse installed capacity was optimized as described in Section 7.2.

(b) Alternative Arrangements

Arrangements P.1, P.2 and P.3 are described below. To facilitate comparison, the arrangements are shown and described based on vertical-shaft Kaplan turbine units.

Arrangement P.1

Arrangement P.1, a cross-channel arrangement shown on Fig. 7-2, is similar to Alternative F.3 and would have the powerhouse located next to the right abutment and the spillway located between the powerhouse and an earth-rockfill embankment dam. The dam would be located in the river channel and an overburden excavation in the left abutment would allow construction of the concrete cut-off wall as an extension to the dam. The concrete structures and channels on the right abutment would require extensive rock excavation. All structures would be oriented along a common axis. The construction sequence would involve excavation and construction of concrete structures behind a cellular cofferdam with the river generally in the existing channel. River diversion through the completed spillway would allow the embankment dam to be built and completion of the powerhouse. Location of the structures would require that the existing highway and rail line be relocated permanently and that the existing transmission lines be relocated temporarily during construction.

Arrangement P.2

Arrangement P.2, a side-channel arrangement shown on Fig. 7-3, is similar to Alternative F.4 and would have the earth-rockfill embankment dam in the river channel as for P.1. A gravity section would be located between the dam and the powerhouse. The powerhouse would be oriented approximately perpendicular to the embankment dam in a downstream direction, and the spillway would be located between the powerhouse and the right abutment at the end of the excavated approach channel. The construction sequence would be similar to P.1 but a cellular cofferdam would not be required. A permanent relocation of the existing highway and rail line would be required but to a lesser extent than for P.1.

Arrangement P.3

Arrangement P.3, shown on Fig. 7-4, is similar to Alternative F.2 except that the axis of the structures has been slightly rotated downstream about the left abutment to tie in with updated bedrock contours. The spillway would be located at the right abutment and the powerhouse would be located between the spillway and the earth-rockfill embankment dam. The construction sequence would involve excavation of channels and construction of the spillway in the dry prior to river diversion. Diversion through the spillway would allow the powerhouse to be constructed (partly in the river channel) and the embankment dam to be built. This sequence would result in a slightly longer construction period than required for P.1. Permanent relocation of the highway and rail line and temporary relocation of the transmission line would be similar to that required for Arrangement P.1.

Preliminary comparative cost estimates indicated that Arrangements P.1, P.2 and P.3 were approximately equal in cost and therefore other factors were considered in the comparison of arrangements.

Impacts on the nearby community of Oasis were compared for Arrangements P.1 and P.2. These impacts included noise, dust and visual aspects of construction and were assessed on the basis of the volume of excavated rock, proximity of excavation limits to Oasis, and offset for highway/rail realignment, which are listed below:

	Arrangement P.1	Arrangement P.2
Volume of rock excavation ($10^6 m^3$)	3.2	2.6
Minimum distance between excavation limits and Oasis	177 m	250 m
Highway/rail offset from existing alignment	100 m	20 m

Comparison on the basis of these parameters favours the side channel arrangement P.2. For this arrangement approximately 75 m of additional buffer area would be provided between the project excavation and Oasis, and existing WKPL transmission lines would not need to be relocated.

Hydraulic performance was also compared for the alternate arrangements. The cross channel arrangement, (P.1), is considered to have superior powerhouse hydraulic performance under all flow conditions but especially during operation of the spillway. It would provide relatively symmetrical flow conditions in the approach and downstream channels.

The small short term environmental advantages of Arrangement P.2 do not offset the potentially important technical (hydraulic) disadvantages which have been identified. As a result Arrangement P.1 was deemed preferable to P.2.

In the comparison between Arrangement P.1 and P.3, the axis for P.1 was also slightly rotated downstream about the left abutment to tie in with updated bedrock contours.

The comparison between Arrangement P.1 and P.3 showed that neither had significant advantages over the other. Although the cost estimates show a slightly lower construction cost for Arrangement P.3, the total capital cost (construction cost plus corporate overhead and interest during construction) would be about the same for both arrangements due to the longer construction period for P.3.

Consideration was given to the comparative locations of the spillway and powerhouse. A powerhouse located next to the right abutment would allow better access during construction and operation. With the spillway located next to the right abutment, a wider and more costly spillway bridge would be required for powerhouse access.

A spillway located separate from the embankment dam is generally preferred in order to minimize potential embankment erosion. However, for the Murphy Project, the earth and rockfill construction materials for the dam, favourable foundation conditions and topography, together with separation by a concrete training wall, is expected to ensure that discharges from the low head submerged discharge spillway would not be detrimental to the embankment dam. The effect of such discharge currents on the embankment dam will be examined during hydraulic model studies proposed for Phase 2.

Based on the various factors described in the foregoing, Arrangement P.1 was selected as the preferred alternative (Section 9.0).

(c) Alternative Turbine Types

Three alternative types of turbine were considered:

Horizontal-shaft Straflo units
Horizontal-shaft bulb units
Vertical-shaft Kaplan units

General arrangements and layouts were prepared for each alternative based on Arrangement P.1 and it was initially proposed to prepare a cost estimate for each. However, the studies for Straflo units were terminated on the basis of similar studies for Keenleyside where it was found that costs for the horizontal-shaft bulb units and Straflo units were similar, but with the understanding that Straflo units could be considered further in Preliminary Design Phase 2 as an alternative to bulb units, should bulb units be selected over vertical-shaft Kaplan units in Phase 1.

The cost estimates prepared for the bulb and Kaplan alternatives were adjusted for the differences in overall plant efficiency. For the horizontal-shaft bulb units (Fig. 7-5) it was estimated that the overall plant efficiency would be about 3 percent lower than for vertical-shaft Kaplan units, with 1.5 percent attributed to the turbine and 1.5 percent to the generator.

Vertical-shaft Kaplan units were found to have an economic advantage over the horizontal-shaft bulb units of about 7 percent of total construction costs excluding corporate overhead and interest during construction and including efficiency adjustments (as a present value).

Based on the above comparisons, vertical-shaft Kaplan units were selected. During Phase 2 Preliminary Design, fixed blade propellor turbines will be compared to Kaplan units.

Prior to issue of turbine tender documents, further studies, including design and layouts, would be undertaken for horizontal-shaft bulb units to permit evaluation of any tenders for this alternative.

7.2 CAPACITY OPTIMIZATION

All Murphy Creek capacity studies were based on maximized net annual benefits[15] and used the following data.

1. Construction Costs - Construction costs (based on October 1981 price levels) were estimated on the basis of the detailed layouts and cost estimates prepared for Kaplan arrangements with four 75 MW units and bulb arrangements with six 50 MW units.

2. Annual Costs - The annual costs were assumed as 4.6 percent of the construction costs. The factor of 4.6 percent was based on a net interest rate of 3 percent and included allowances for interest, depreciation, corporate overhead, interest during construction plus operating and maintenance costs. Annual water license fees or rentals and property taxes were not included.

3. Annual Benefits - Energy and capacity values at the powerplant site (based on October 1981 price levels) were as follows:[15]

Firm energy	23.0 mills/kW.h
Secondary energy	17.2 mills/kW.h
Dependable capacity	$9.33/kW/yr

Firm energy is the energy that could have been produced during the critical drought on the B.C. Hydro system, which corresponds to the historic streamflow record between September 1942 and April 1946.

Secondary energy is the 35-year (1940-1975) average annual energy minus the firm energy.

Dependable capacity is the capacity available with reservoir conditions corresponding to January 1945 streamflow.

Benefits allow for energy losses at Brilliant Dam. (Energy losses at Brilliant are governed by Murphy reservoir levels and are not affected by installed capacity at Murphy.)

Capacity optimization studies were initially undertaken on the basis of Operating Regime 1 and the installed capacity was selected as 260 MW with a powerhouse having four vertical-shaft Kaplan units.

Following the adoption of Operating Regime 2 for power studies, an optimization study was carried out for horizontal-shaft bulb units and vertical-shaft Kaplan units combined with Arrangements P.1 and P.2.

Installed capacity for each combination was optimized based on maximizing net annual benefits up to the maximum permissible turbine size. The physical limitations assumed for runner diameter were 8.5 m for vertical-shaft Kaplan units and 7.5 m for bulb units.

The optimization study showed that 300 MW was the optimum capacity for the Kaplan units with Arrangement P.1. Net annual benefits versus installed generating capacity for the Kaplan units with Arrangement P.1 is shown on Fig. 7-6.

The optimum capacity of 300 MW corresponds to a hydraulic capacity of 2620 m^3/s. Maximum net annual benefits for this capacity would be about $20 million, obtained with five units. With four units and an installed capacity of 260 MW, net annual benefits would be about $19.5 million. As net annual benefits are close for these alternatives other factors were considered prior to making a recommendation.

1. Five Kaplan units (300 MW) develop about 185 GW.h more energy than four Kaplan units (260 MW). This represents the difference between obtaining 87 percent of the maximum energy potential at the site with four units and 96 percent with five units.

2. There is a possibility that the Brilliant powerplant will be expanded. A five Kaplan unit arrangement would have a better hydraulic balance with an expanded Brilliant powerplant than a four unit arrangement.

3. The optimum capacity of 300 MW with a hydraulic capacity of 2620 m^3/s can only be achieved with five or more Kaplan units.

Based on the above studies, a five-unit vertical shaft Kaplan arrangement was selected with an installed capacity of 300 MW (Section 9.0). The unit ratings are:

Rated generator output	60 MW (66.7 MV.A @ 0.9 pf)
Rated turbine output	61.2 MW
Rated net head	13.0 m
Rated hydraulic discharge	524 m^3/s

SECTION 8.0 - RESERVOIR

8.1 GENERAL

The reservoir would extend to the Hugh Keenleyside Dam on the Columbia River and Brilliant Dam on the Kootenay River (Fig. 1-2). With the normal operating reservoir level at El. 424 m, the Murphy Creek Project would raise the water level approximately 15 m at the project site, 4 m at the Hugh Keenleyside Dam and 3 m at the Brilliant Dam. The total flooded area above existing mean high water level would be approximately 300 ha. The area of the reservoir at the normal operating level (El. 424 m) would be about 1300 ha. The volume of the reservoir at this level would be about 140 x $10^6 m^3$. The reservoir area and volume curves are shown on Fig. 8-1.

The existing riverbanks in the project area consist of granular materials including silt, sand, gravel, cobbles and boulders and in some areas these materials stand relatively steeply (1.3 H to 1 V). As the toes of these banks were eroded, flatter slopes were developed near the present river level. In some areas gravel, cobbles and boulders provide natural armouring for the existing flatter slopes.

Studies have been made to assess the impact of the reservoir on the shoreline.[24] Terrain analyses indicate that the main cause of any shoreline erosion following reservoir filling would be beaching due to wave action and regression. The possibility of deep-seated sliding due to reservoir filling and normal drawdown is considered remote. Based on this conclusion, shoreline regression has been estimated using beach slopes for the various materials along the proposed reservoir shoreline. The potential beach slopes were determined using information obtained from the Arrow, Mara and Whatshan Lakes in B.C. A study of wind duration at various reservoir elevations indicated that beach erosion would occur with the reservoir near the normal operating level (El. 424 m).

Formation of beach slopes would generally be accompanied by regression of the banks behind the beach until a stable condition is reached. The amount of regression of the banks behihd the beach was based on developing a generalized stable slope assumed to be 1.5 H to 1 V. From this assumed mode of regression, a preliminary safeline, defined below, was developed without consideration of bank protection.

> "The safeline is a line around the reservoir outside of which the security of residents and their belongings can be reasonably assured.
>
> The location of the safeline is determined by the application of geotechnical and hydraulic criteria. The safeline allows for maximum potential regression of the bank around the reservoir. In areas where bank regression is not involved freeboard allowances govern the location of the safeline."

The shoreline has been studied to determine which portions would be protected against erosion.[26] The study included an economic evaluation by comparing preliminary estimated costs of the protective works to the value of property that would be protected, and was based on the objective to keep reservoir impacts on property to a minimum. In most areas where the preliminary safeline without protection would otherwise impact on residential and industrial buildings, rail lines, cemeteries, roads, reservoir crossings and sewage lagoons, shoreline protective works are proposed. These protective works would limit the impact of the reservoir.

Where shoreline protective works are proposed, they would be provided before reservoir filling in most locations. Some roads would be protected after reservoir filling if significant erosion occurred. The protective works would be rock and/or granular fill toe protection along the banks of the river.

Generally flat areas at or near the reservoir level could possibly be reclaimed by berms and fill. However, final assessment of such areas has not been completed. These will be resolved after further studies by Hydro and consultation with the public, Municipalities and Regional Districts involved.

Fig. 8-2 shows the location of the proposed shoreline protective works and possible berm and fill areas.

In some low-lying residential areas there are basements and services which could be affected by high groundwater levels. In these areas groundwater could possibly be lowered or modifications could possibly be made to the residences. However, final assessment of such areas has not been completed. These will be resolved after further studies by Hydro, and consultation with the public, Municipalities and Regional Districts.

Construction of the proposed dam and impounding of the reservoir could affect the following facilities:

1. Infrastructure, including municipal water supply and sewage, storm drains and roads.

2. Utilities, including gas pipelines, telephone lines and electric transmission lines, both crossing the river and on the shoreline.

3. BC Timber Pulp and Sawmill.

4. C.P. Rail line, including bridges.

5. Highways, including bridges.

Facilities studied include: utilities crossing the river and along the shoreline, the relocation of the provincial highway and C.P. Rail line, and the temporary relocation of the West Kootenay Power and Light Ltd.

(WKPL) transmission lines, around the project structures. These studies included input from the respective government agencies, private companies and utility.

Consultants to B.C. Hydro have assessed the affect of the Project on the C.P. Rail facilities,[25] Municipal infrastructure[27] and the BC Timber Pulp & Sawmill.[28]

8.2 SHORELINE PROTECTIVE WORKS

Protective works would be constructed in the locations shown on Fig. 8-2, to reduce the effect of the reservoir on residential and industrial buildings, railway, sewage lagoons, cemeteries and roads.[26]

Two types of protective works have been studied, toe protection and protective berm and fill. Toe protection has been studied as a proposed mitigation measure. Protective berm and fill has been studied as a possible mitigation option.

Toe protection would involve the placement of materials at the toe of existing slopes which would prevent beaching and regression of the riverbanks. Toe protection would generally be placed prior to reservoir filling except in certain areas where slopes would be monitored and toe protection provided after reservoir filling, if significant erosion occurred.

Three typical sections for toe protection are shown on Fig. 8-3, using various combinations of sand/gravel and riprap (rounded boulders or angular rock).

Protective berm and fill would involve the placement of granular materials on certain relatively flat areas that are close to the reservoir level. The existing ground level could be raised by filling to form a flat area at an elevation above that which would be affected by the reservoir.

A section of a berm and fill area is shown on Fig. 8-2, using sand and gravel with rockfill protection on the reservoir side. In some areas where fine materials exist a sand filter may also be required.

There are some relatively flat non-residential areas where the existing ground elevation is at or slightly above El. 425 m with local depressions below El. 424 m. Most of these areas are located near Brilliant. These depressions could be filled to El. 425 m to ensure the entire area would be above reservoir level.

The crest elevation of the protective works was chosen to incorporate a freeboard allowance added to the normal operating level, El. 424 m, or to the 200-year flood level (Fig. 5-2), whichever was greater. The freeboard allowance was based on projected wave run up, use of property to be protected, slope of adjacent ground and the degree of exposure to wind-generated waves.

An analysis of wind-generated waves was carried out to determine the wave run-up at different locations on the reservoir. Since the reaches were generally found to have a limited fetch, an idealized channel was chosen for design and the wave heights calculated for the design wind. Run-up on the shore was calculated for a range of slopes.

The size of material required for the reservoir side of the protective works was based on the design wave height, nature of protective materials and slope.

Additional studies on protective works including methods and materials for construction will be undertaken during Phase 2 of the Preliminary Design studies.

8.3 INFRASTRUCTURE

Municipal infrastructure has been studied to determine what remedial works would be necessary to rehabilitate or replace facilities which would be affected by the reservoir.[27] Infrastructure that would be affected includes water supply and sewage works, storm drainage and local roads and is located in Robson, East Raspberry, Ootischenia, North Castlegar, South Castlegar and China Creek (Fig. 8-4).

A summary of municipal infrastructure that would be affected together with proposed remedial measures would be:

Location	Infrastructure	Proposed Remedial Works
Robson/Raspberry	Road	Raise
	Sewerage	Modifications to sewage lagoon effluent works Replace drain fields
North Castlegar	Roads	Raise
	Sewerage	Relocate pump stations
	Drainage	Replace
	Waterworks	Relocate watermains
South Castlegar	Roads	Raise
	Waterworks	Raise well and main.
	Sewerage	Add diffuser, relocate pump stations
Genelle	Roads	Raise
	Waterworks	Reconstruct
	Sewerage	Reconstruct

8.4 UTILITIES

Utilities which would be affected by the reservoir are:

1. WKPL transmission lines (main and distribution).

2. B.C. Telephone; telephone/cable lines.

3. C.P. Telecommunications; cable lines.

4. Inland Natural Gas; pipeline.

In the project area, utilities cross the reservoir and are located along the shorelines. The impacted crossings would be relocated and constructed to conform to regulatory clearance requirements. Towers, poles and other structures would be relocated beyond the preliminary reservoir impact line. There are 14 utility crossings of the Columbia River in the project area, of which 12 would be impacted. These are shown on Fig. 8-5.

The buried Inland Natural Gas pipeline in North Castlegar would be submerged in certain areas. The buried pipeline would be protected, weighted to counteract buoyancy, and valves and controls raised (Fig. 8-4).

The reservoir would inundate a portion of the West Kootenay Power & Light transmission line which runs adjacent to the shoreline at Waterloo Eddy, just downstream of Ootischenia. The inundated poles would be raised on fill and this fill protected from erosion (Fig. 8-4).

A section of the 8-pole 60 kV transmission line owned by West Kootenay Power & Light and Cominco, located near the project site, would be temporarily relocated during construction. Following relocation of the highway and rail line, the transmission line would be rebuilt along its original right-of-way. The poles would be located to provide adequate clearance for crossing of the highway and rail line (Fig. 9-12).

8.5. BC TIMBER PULP AND SAWMILL

The reservoir would affect some of the existing shoreline structures, protective works and unprotected riverbanks at the BC Timber Pulp and Sawmill. The existing protective works would be upgraded and some of the existing unprotected banks and structures would be protected.[28]

8.6 C.P. RAIL LINE

The excavation for the Murphy Creek Project structures would impact the C.P. Rail line. The railway line would be relocated between the excavation limits and the relocated highway (Fig. 9-12).

Filling of the reservoir would impact approximately 5.4 km of track between Sullivan Creek and the Project site. The proposed remedial works would include raising and/or realigning and protecting the railway track while maintaining rail traffic, rebuilding the Murphy Creek crossing and the Birchbank siding and constructing a retaining structure near the Murphy Creek crossing[25] (Fig. 8-4).

Upstream of Sullivan Creek there are areas where the reservoir could affect the C.P. Rail line. In these areas protective works would be placed along the toe of the slopes. Protective works would also be placed along the toe of the slopes beneath the relocated track between Birchbank and the Project site. Protective works are also proposed for the C.P. Rail bridge crossing in North Castlegar (Fig. 8-5).

8.7 HIGHWAYS AND ROADS

Excavation for the Murphy Creek Project structures would impact the highway between Trail and Castlegar. The highway would be relocated between the relocated C.P. Rail line and the community of Oasis (Fig. 9-12).

The highway between Robson and the Keenleyside Dam could be affected along portions of its length by beaching of the reservoir. The bank on the reservoir side of the road would be monitored after impounding and protective works would be placed as required if significant erosion occurred. The area is accessible from the highway for placing of protection materials (Fig. 8-4).

The gravel road between Ootischenia and Champion Creek would be affected by the reservoir over a length of approximately 2 km. A length of about 0.9 km would require toe protection and approximately 1.2 km would require raising and relocation.

Some localized protective works would be required for the abutments of the highway bridge in Castlegar (Fig. 8-5).

SECTION 9.0 - PROPOSED PROJECT ARRANGEMENT

9.1 GENERAL

The proposed project arrangement is shown on Fig. 9-1. It would comprise an earth-rockfill dam with till core and concrete cut-off wall in the present river channel, a powerhouse located on the right bank housing five vertical-shaft Kaplan units and a five-bay spillway between the powerhouse and the dam. The existing highway and railway on the right bank would be moved to the west.

The major project features, including the cellular cofferdam, are described in this section. The installed capacity of 300 MW was selected on the basis of an optimization study (Section 7.0). Further refinement of the individual project components will be carried out during Phase 2 of the Preliminary Design Studies using updated economic and cost criteria.

Guidelines adopted for the preliminary design are given in Appendix A. They cover the load cases, water levels, mass of materials and uplift conditions to be used for stability analysis. These guidelines are consistent with the design criteria used for other projects designed by B.C. Hydro.

9.2 COFFERDAMS

Diversion would take place in two stages (Section 10.2). First-stage diversion would channel the river toward the left bank using a cellular cofferdam. Second stage diversion would be through the spillway. Areas unwatered for spillway and powerhouse construction would be protected by rock plugs. The dykes used to close the river channel in second stage diversion would be incorporated into the toes of the embankment dam.

The cellular cofferdam with earth-rockfill wing dams and a crest at El. 416 m, would be used to direct the Columbia River to the left side of the existing river channel during the first stage of diversion (Fig. 9-2). The cellular cofferdam portion would be about 240 m long, comprising 15 cells each about 15.5 m in diameter. An earth-rockfill wing dam would be constructed at each end of the cellular cofferdam. A plan and sections of the cofferdam are shown on Fig. 9-2. The cells would be filled with well-graded sand and gravel to provide high unit weight and high internal friction to increase the cell stability.

The location of the cofferdam has been selected to minimize the excavation required for the concrete structures and approach and tailrace channels. The river channel would be widened by excavating a portion of the left bank, and protecting it with riprap, to provide an adequate channel to keep river levels below the top of the cofferdam (Fig. 9-1).

For the diversion design flood of 6310 m^3/s the water level upstream of the cofferdam would be at El. 415.7 m and the tailwater would be El. 414.1 m. Flow velocities would be subcritical thoughout the reach. The final design for the diversion channel will be investigated on the proposed comprehensive hydraulic model (Section 5.5). Since a flood with the magnitude of the diversion design flood would be forecast in advance of the event, the fill on top of the cellular cofferdam could be raised, if necessary, for extra protection. The effect of postponed filling of the Mica Reservoir on the flood peak, as noted in Section 4.2, will also be investigated further.

It is estimated that up to 11 m of riverbed material exists in the cofferdam area. The riverbed material is granular and could be locally bouldery and very pervious. To reduce seepage through this material, the sheetpiling for the cofferdam cells would be driven through the overburden to bedrock. Driving of sheetpiling would be difficult in areas where the riverbed material is bouldery. To minimize construction problems, the locally existing bouldery overburden material could be excavated to rock surface prior to cellular cofferdam construction. Based on preliminary drilling information, the maximum heights of the cellular cofferdam above rock would be about 27 m on the upslope side and 19 m on the downslope side.

9.3 EMBANKMENT DAM

The embankment dam would be a zoned earth-rockfill dam consisting of a central till core with a concrete cut-off wall, rockfill and sand and gravel shells, filter and transition. The earth-rockfill dykes used in the second stage diversion, described in Section 10.2, would be incorporated into the toes of the embankment dam (Fig. 9-3). The embankment dam would be about 150 m long and up to 27 m high. The crest of the dam would be at El. 427.0 m providing freeboard of about 3 m under normal operating and PMF conditions. An analysis of wind-generated waves was also carried out to ensure that the selected freeboard was sufficient to prevent overtopping from waves with occurrence probabilities approaching the PMF. The upstream and downstream slopes of the dam would be 2 H:1 V and 1.8 H:1 V respectively.

The foundation of the embankment dam would be on granular overburden material in the river bottom and on the left abutment. Exploratory drilling has indicated the possible existence of a deep, rock depression (buried river channel) under the left bank of the river. The material infilling this channel is very pervious with a permeability estimated to be about 10^{-1} cm/s. To avoid damage or failure of the earthfill dam that could occur due to migration of material under the dam and to avoid excessive loss of water, a concrete cut-off wall to bedrock would be constructed across the buried channel and into the left abutment.[29]

9.4 SPILLWAY

The spillway would be located between the embankment dam and powerhouse and would have a maximum discharge capacity of 13 100 m^3/s, the flow resulting from the Probable Maximum Flood (PMF). With this flow the water level at the dam would be El. 424.2 m (dam crest at El. 427 m).

The spillway would comprise a five-bay gated concrete overflow structure 107 m wide and 40 m long and a concrete stilling basin 60 m long (Fig. 5-1). The spillway piers would be 4.5 m wide. The pier adjacent to the embankment dam would be incorporated in a gravity retaining wall. The spillway radial gates, each 16 m wide and 21 m high, would be operated by hydraulic cylinders. Slots would be provided for installation of stop-logs upstream and downstream of the gates to facilitate inspection and maintenance. The upstream stop-logs would be placed from a roadway deck by the intake gantry crane. The downstream stop-logs would be placed by barge.

The stilling basin would be of conventional design with invert (El. 399 m) about 1 m below the bottom of the existing river channel and would contain the hydraulic jump for the 200-year flood of 7590 m^3/s. For larger floods some erosion of the riverbed downstream would be acceptable. Foundation drainage would be provided by a line of drainholes drilled from the drainage and grouting gallery. The upper foundation rock may require some grouting.

9.5 POWERPLANT

(a) Powerhouse Structure

The powerhouse, with five 60 MW Kaplan turbines, would be a conventional low head indoor surface structure (Figs. 9-4 to 9-11) and would be about 201 m long.

A steel superstructure was selected as the most economic compared with cast-in-place reinforced concrete. Cladding would consist of pre-cast hollow core, insulated concrete panels. This type of cladding would provide the building with efficient thermal insulation.

A 46 m long by 28 m wide service bay, situated at the west end of the powerhouse at the right bank of the river, would provide sufficient space to accommodate simultaneous assembly of two generator rotors during construction.

Each unit would have intakes consisting of three passages separated by piers. Trashracks would be provided at the upstream end of intake piers.

Foundation drainage, similar to that for the spillway, would be provided.

(b) Gates and Cranes

The intake gates would include two sets of three bulkhead gates for closure of two units and five sets of three hydraulic operating gates for the five units.

Servicing of the intake gates would be carried out using an 80-tonne gantry crane. A jib with 0.5 m^3 capacity hydraulic grapple would be provided on the upstream side of the gantry crane, for cleaning the trashracks.

For draft tube closure, two sets of three gates for two units would be provided. Operation of the draft tube gates would be achieved by using a 60-tonne gantry crane.

The generating units would be serviced by two 275-tonne bridge cranes, operating throughout the full length of powerhouse.

(c) Turbines

The five 60 MW vertical shaft Kaplan turbines, with runner diameter of approximately 8.5 m, would operate at 63.2 rpm and develop their rated output of 61 200 kW at a net head of 13 m. The rated discharge of each unit would be approximately 524 m^3/s.

(d) Generators

Each of the five turbines would be coupled to umbrella or semi-umbrella type generators with synchronous speed of 63.2 rpm and rated 60 MW at 13.8 kV, 0.90 power factor. The generators would be connected by naturally cooled isolated phase bus to the three phase generator transformers rated 67 MVA, 13.8 kV to 230 kV installed on the tailrace deck.

(e) Transformers and Switching Equipment

Generator circuit breakers would be installed between each generator and its associated transformer to provide protection and/or synchronization of the generator. These circuit breakers would be installed in a downstream gallery below the tailrace deck (Fig. 9-4).

The 230 kV metalclad SF6 insulated switchgear would connect the generator transformers to the outgoing 230 kV transmission lines, which would be attached to the downstream side of the powerhouse. The switchgear, consisting of circuit breakers, disconnect switches, ground switches and bushings would be located on the tailrace deck.

The station service would be provided at 13.8 kV from taps off the isolated phase bus. A standby diesel generator would be installed to permit operation of control gates if there is loss of normal electrical power.

(f) Control

The operation of the powerhouse would be controlled from the local control centre and the system control centre on Burnaby Mountain.

(g) Water Supply System

Because of the low operating head at Murphy Creek, a pumped water supply system must be provided for fire protection, turbine and generator cooling and domestic water. For this purpose, a raw water pump room would be located under the service bay at El. 410.5 m.

9.6 HIGHWAY, RAILWAY AND TRANSMISSION LINE RELOCATION

To accommodate the layout of the project structures, the existing highway and railway would have to be relocated around the powerhouse/service bay, and the WKPL transmission line would have to be temporarily relocated.

The general arrangement for the relocated highway, railway and transmission line is shown on Fig. 9-12.

The relocated highway would be designed to meet the standards of the B.C. Ministry of Transportation and Highways for a design speed of 80 km/h.

Design criteria for railway relocation are shown on Fig. 9-12.

During construction of the project and realignment of the highway and railway, the transmission line would be temporarily relocated. Following construction, the transmission line would be re-installed on the existing right-of-way with poles relocated as required to suit the highway and railway revised alignments.

The access road to the completed dam crest would cross the railway by level crossing. The access road to the powerhouse would pass under the railway.

SECTION 10.0 - CONSTRUCTION

10.1 CONSTRUCTION SCHEDULE

The proposed construction schedule for the five 60 MW Kaplan units cross channel arrangement is shown on Fig. 10-1.

The schedule provides for the following in-service dates, which are as stipulated in B.C. Hydro's 1983 Electric System Plan:[10]

Unit 1	July	1993
Unit 2	October	1993
Unit 3	February	1994
Unit 4	June	1994
Unit 5	October	1994

The overall construction period from award of first contract, in February 1988, to the first unit in-service in July 1993 would be 5 1/2 years. With subsequent units coming on line at approximately 4-month intervals, the total project construction period for all work, including site restoration, would be 7 years.

The river would be diverted through the spillway in December 1991, approximately 4 years after start of work and 15 months before reservoir filling in March 1993.

Activities affected by river levels, such as sheet pile installation, river channel widening, and rock plug removal, have generally been scheduled during the spring and fall low flow periods.

River diversion facilities and sequences have been planned to allow powerhouse construction and equipment installation to proceed independantly of other construction activities. One critical path for project activities goes through construction of the powerhouse and installation of the generating equipment. A second critical path goes through the following activities: cellular cofferdam - spillway construction - river diversion - embankment dam construction - reservoir filling.

The site preparation and first stage diversion work would be awarded no later than February 1988.

The turbine and generator contracts would be awarded in February 1988, 16 1/2 months prior to award of the powerplant contract, so that the dimensions of the powerhouse water passages could be finalized with the turbine contractor prior to the issue of tender documents for the powerplant contract.

The main civil contract for construction of the powerplant would be awarded in July 1989, 48 months prior to the first unit in-service date.

The current engineering schedule for the Murphy Creek Project provides for completion of Preliminary Design Phase 2 in December 1984 and start of Final Design in April 1985.

10.2 CONSTRUCTION SEQUENCE

(a) River Diversion

For construction of the proposed arrangement, the river would be diverted in two stages: the first stage diversion is required for construction of the spillway, the second stage diversion is necessary for construction of the embankment dam.

During the first stage diversion (Fig. 10-2, Phases B and C) the river would be channelled towards the left bank by the cellular cofferdam.

Construction of the powerhouse and spillway would proceed in the unwatered area protected by the cellular cofferdam and spillway and powerhouse rock plugs.

During the second stage diversion (Fig. 10-2, Phase D) the river would flow through the spillway. The powerhouse structure would be protected from the river by the spillway west pier and training wall and the powerhouse rock plugs.

(b) Construction Phases

The construction sequence can be divided into five main phases, as shown on Fig. 10-2 and as follows:

Phase A - Site Preparation (January 1988 to March 1989)

Following temporary relocation of the WKPL transmission lines, project construction work would start in April 1988 with relocation of Highway 22 and the C.P. Rail line away from the powerhouse area, development of site access, camp, work areas, and first stage excavation of the spillway and powerhouse approach and discharge channels.

Immediately after construction of a temporary construction bridge across the Columbia River, the river channel would be widened on the left bank in preparation for the first stage diversion.

Phase B - First Stage Diversion (February 1989 to February 1990)

Construction of the cellular cofferdam and wing dams would start in March 1989 and be followed by excavation for the spillway and gravity wall structures, scheduled to start in November 1989 after unwatering of the area inside the cellular cofferdam.

Excavation for the approach and discharge channels would continue during this phase. Excavation for the powerhouse and intake structures would commence in September 1989 and be completed by June 1990.

Mobilization for aggregate processing and concrete batch plant would start in September 1989.

Phase C - Powerplant Construction (March 1990 to September 1991)

Concrete placement would start in March 1990 initially for the powerhouse, and soon after for all other structures.

The spillway structure and gates would be completed by September 1991, ready for second stage diversion.

Excavation for the approach and discharge channels would be completed inside the upstream and downstream rock plugs. Excavation of riverbed material and bedrock outside the rock plugs would also be completed prior to September 1991.

Excavation of overburden material on the left bank upper bench would be carried out so that construction of the first section of the concrete diaphragm wall could start in April 1991.

Phase D - Second Stage Diversion (October 1991 to September 1992)

Removal of the cellular cofferdam and wing dams would start in October 1991 and rockfill dykes would be placed across the river to divert it through the spillway before the winter.

Granular material would be dumped and compacted between the rockfill dykes to form a work platform for the concrete diaphragm wall section below the embankment dam core. Construction of this portion of cut-off wall would start in April 1992 and be complete by September 1992.

Powerhouse construction and installation of gates and equipment would continue in the unwatered channel behind the powerhouse rock plugs.

Phase E - Completion (October 1992 - December 1994)

Removal of the upstream and downstream powerhouse rock plugs would start no later than October 1992 after installation of the intake gates and draft tube gates.

Construction of the embankment dam would be completed before winter 92/93, ready for reservoir impounding in the spring of 1993 and testing of the first generating unit.

Installation of all remaining powerhouse equipment would be completed in the summer of 1994.

Concrete diaphragm wall construction on the left bank would be completed in the spring of 1993.

Removal of construction facilities and site rehabilitation would be essentially completed by the end of 1994.

10.3 MAJOR CONTRACTS AND WORK QUANTITIES

To perform the work as shown on the proposed construction schedule the following major contracts would be considered:

1. An initial civil contract (MC1) for site preparation, temporary bridge over the Columbia River, cellular cofferdam and first stage channel excavation, to be awarded in February 1988 and completed in October 1989.

2. A Contract (MC2) for the supply and installation of turbines to be awarded in February 1988 and completed in October 1994.

3. A Contract (MC3) for the supply and installation of generators to be awarded in February 1988 and completed in October 1994.

4. A main civil contract (MC4) for completion of all excavation, construction of the powerhouse and spillway structures, installation of auxiliary equipment, removal of cellular cofferdam and embankment dam construction, to be awarded in July 1989 and completed in December 1994.

5. A contract (MC5) for concrete diaphragm wall construction, to be awarded in February 1991 and completed in April 1993.

Major quantities for the civil work are as follows:

Contract MC1

Right bank, first stage excavation (rock)	1 100 000 m^3
Left bank, river channel excavation (overburden)	350 000 m^3
Cofferdam and sheet pile cells fill	100 000 m^3

Contract MC4

Right bank, completion of excavation (rock)	1 700 000 m³
Left bank, seepage cut-off excavation (overburden)	750 000 m³
Embankment dam fill	180 000 m³
Powerplant concrete	300 000 m³

Contract MC5

Concrete diaphragm wall	22 000 m²

10.4 CONSTRUCTION METHODS AND FACILITIES

(a) Construction Methods

The methods and equipment used for construction of the project would be of the Contractors' choice, subject to B.C. Hydro's approval. However, for planning purposes, certain assumptions have been made as to the construction methods.

For excavation and concrete work, conventional methods and production rates achieved on similar B.C. Hydro projects were assumed. Some underwater drilling would be required for riverbed materials outside the approach and discharge channel rock plugs. Rock plugs have been assumed to be left in place for unwatering construction areas.

The concrete cut-off wall would be a panel wall constructed using tremie concrete placed in a slurry trench as described in Section 9.3 and Ref. 29. Methods and production rates assumed for the preparation of the cost estimate and construction schedule were based on information furnished by firms specialized in such work.

The sequence assumed for placement of powerhouse concrete and installation of turbine parts was based on information obtained from other utilities and suppliers who have had experience with vertical Kaplan unit installations.

(b) Borrow and Spoil Areas, Construction Facilities

The proposed site arrangement is shown on Fig. 10-3. Initial disposal areas for the approximate 2 800 000 m^3 of channel excavation material would be upstream of the project on the right bank, with subsequent disposal by hauling across the construction bridge to the Billy Creek spoil area. The creek would be channelled through a culvert. Approximately 100 000 m^3 of rockfill would be stockpiled for construction of the embankment dam. Temporary channel widening on the left bank of the river and excavations for the concrete diaphragm wall would result in approximately 1 100 000 m^3 of material which would be spoiled at Billy Creek.

The source of approximately 22 000 m^3 of till for the cofferdams and embankment dam core would be the proposed borrow area on the slopes above Murphy Creek (Area 4).

The proposed borrow area for approximately 540 000 m^3 of concrete aggregate and granular materials is on the left bank, immediately upstream of the project structures (Area 7). The aggregate plant would be located in the borrow area, and a concrete batch plant would be located on the right bank near the construction bridge.

A rail siding and warehouse would be provided on the work area upstream of the approach channel for delivery and storage of project equipment.

The contractor's offices and buildings would be located primarily in the work area downstream of the discharge channel, and

B.C. Hydro offices would be at the riverfront edge of the camp area.

10.5 CONSTRUCTION CAMP

The proposed location for the single labour camp is on the right bank, upstream of Hanna Creek.[30] The camp capacity would be 600 men (60% of peak force) and it would consist of eight 2-storey bunkhouses, with kitchen-dining hall and recreation hall.

A water supply system would be installed for the camp, and a sewage treatment system would be provided for effluent from the camp. Power for the site would be tapped off the WKPL transmission lines near the site.

10.6 ACCESS ROADS

The main access to the camp and project site would pass under the railway from the relocated section of Highway 22 below Oasis. The access road to the completed dam crest would cross the railway by level crossing.

For construction purposes, a temporary bridge would be provided across the Columbia River at Billy Creek, mainly for the purpose of hauling surplus rock from the right bank excavation to Billy Creek spoil area, and to transport aggregate from the left bank aggregate pit for concrete placement in the right bank structures. This bridge would be removed in 1993 when access is possible across the dam. No public traffic would be allowed over this bridge.

10.7 CONSTRUCTION IMPACTS

The communities of Oasis and Rivervale are located close to the proposed location of the project. During construction there would be major activities in rock excavation and hauling to spoil areas. From these activities as well as general earth moving operations, there would be a potential impact on the adjacent communities due to noise, vibration and dust. A study[31] has identified these impacts and the effect on the communities. Studies have also considered possible ways to reduce the impacts.

SECTION 11.0 - PROJECT COST ESTIMATES

11.1 CONSTRUCTION COSTS

The estimate of total generation project construction costs excluding inflation is given in Table 11-1. Costs are shown at October 1982 price levels, except for expenditures incurred in prior years.

Costs estimates are included for land acquisition, flowage, royalties, studies, mitigation and compensation.

Corporate overhead and interest during construction are excluded from the total construction cost.

The total construction costs of the selected project arrangement, at October 1982 prices, have been estimated as:

Total generation project construction cost = $632 million
Cost of capacity = $2107/kW

Table 11-2 shows the rate of expenditure by fiscal years during the engineering and construction period and is based on the construction schedule (Fig. 10-1).

11.2 ESTIMATE OF LABOUR FORCE

Table 11-3 shows the estimated labour force by fiscal years. This is based on the rate of expenditures by fiscal years and the construction schedule.

REFERENCES

1. Montreal Engineering Company Ltd., British Columbia Energy Board Provincial Power Study, Volume 6 Appendix VII, April 1972.

2. Hydroelectric Design Division, British Columbia Hydro, Murphy Creek Project Prefeasibility Study Report No. 939, November 1978.

3. Hydroelectric Generation Projects Division, B.C. Hydro, Murphy Creek Project, Feasibility Study, Report No. H 1335, March 1981.

4. Hydroelectric Generation Projects Division, B.C. Hydro, Hugh Keenleyside Powerplant, Prefeasibility Study, Report No. H 1374, July 1981.

5. Hydroelectric Generation Projects Division, B.C. Hydro, Columbia River, Murphy Creek Project, Engineering Description, Report No. H 1565, November 1982.

6. Hydroelectric Generation Projects Division, B.C. Hydro, Columbia River, Keenleyside Powerplant Project, Engineering Description, Report No. H 1560, November 1982.

7. System Engineering Division, B.C. Hydro, Economic Evaluation of Alternative Murphy Hydro Developments, Report No. SE 8113, July 1981.

8. McDaniels Research Ltd. Keenleyside-Murphy Project, Benefit/Cost Analysis, Draft Report, February 1983.

9. Hydroelectric Generation Projects Division, B.C. Hydro, Columbia River, Keenleyside Powerplant Project, Preliminary Design Study, Phase 1, Report No. H 1613, May 1983.

10. System Engineering Division, B.C. Hydro, 1983 Ten-Year Electric System Plans and Evaluation (Summary Outline), 1 April 1983 to 31 March 1993, December 1982.

11. Hydroelectric Generation Projects Division, B.C. Hydro, Murphy Creek Project, Hydrology, River Morphology and Climate, Report No. H 1350, 1982.

12. Hydroelectric Generation Projects Division, B.C. Hydro, Murphy Creek Project, Flood Studies, Volume I: Derivation of the Probable Maximum Flood, Report No. H 1233, May 1981.

13. Hydroelectric Generation Projects Division, B.C. Hydro, Murphy Creek Project, Flood Studies, Volume II, Derivation of the Regulated Flood Frequency Curve, Report No. H 1598, March 1983.

14. Pacific Hydrology Consultants Ltd., Murphy Creek Project, Preliminary Design, Groundwater Studies, January 1983.

15. Internal Memorandum from D.G. McFarlane to F.J. Patterson, 19 March 1982.

16. B.C. Engineering Company Ltd., Report on an Investigation of the Columbia River in Canada, 1958.

17. Dolmage, V., Columbia River Study, Geological Report, Murphy Dam Site, 27 November 1957.

18. Hydroelectric Generation Projects Division, British Columbia Hydro, Murphy Creek Project, Feasibility Study, 1979/80 Geotechnical Investigations, Report No. H 1162, January 1981.

19. Hydroelectric Generation Projects Division, British Columbia Hydro, Murphy Creek Project, 1981 Geotechnical Investigations, Report No. H 1533, August 1982.

20. Hydroelectric Generation Projects Division, British Columbia Hydro, Murphy Creek Project, 1982 Geotechnical Investigations, Report No. H 1597, March 1983.

21. Klohn Leonoff Ltd., Murphy Creek Project, Hydrogeological Study, PB 7129-0201, March 1983.

22. Thurber Consultants Ltd., Murphy Creek Project Terrain Analysis, 1980.

23. Hydroelectric Generation Projects Division, B.C. Hydro, Murphy Creek Project, Geology and Landforms, Report H 1280, March 1983.

24. Thurber Consultants Ltd., Murphy Creek Reservoir Shoreline Assessment.

25. Thurber Consultants Ltd., Murphy Creek Project, Impact on Existing CP Rail Facilities, June 1982.

26. Hydroelectric Generation Projects Division, B.C. Hydro, Reservoir Shoreline and Proposed Protective Works Report H 1616, May 1983.

27. Dayton and Knight Ltd., Murphy Creek Project, Phase II Municipal Studies, June 1982.

28. Stothert Group Inc., Murphy Creek Project, Study of Impact on BC Timber Facilities, December 1980.

29. Hydroelectric Generation Projects Division, B.C. Hydro, Preliminary Design Study Memorandum on Embankment Dam Foundation Seepage Control, Report H 1604, 1983.

30. Bowman Engineering Associates Ltd., Murphy Creek Project, Camp and Office Development Study, Preliminary Design Report, 1983.

31. Harford, Kennedy, Wakefield Ltd., Murphy Creek Hydroelectric Project, Assessment of Noise and Vibration from Construction, 1982.

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This report was reviewed by HGPD Section Supervisors and Department Managers and by the Murphy Creek project team members outside HGPD.

TABLE 6-1

MURPHY CREEK PROJECT

CONSTRUCTION MATERIALS INVESTIGATION AREAS

Area	Location	Material Type	Year Investigated*1	Scope of Investigation	Remarks
1	Damsite	Granular fill	1981	Field mapping	Overlaps Area 7
2	Hanna Creek	Impervious till	1981	Auger drilling and test pitting	
3	Hanna Creek	Impervious till	1981	Test pitting	
4	Murphy Creek	Impervious till*2	1981 1982	Test pitting Auger drilling	Proposed source for impervious till
5	Topping Creek	Impervious till	1981	Test pitting	Material variable
6	Murphy Creek	Granular fill	1980/81	Becker drilling test trenches and test pitting	Murphy Creek fan
7	Damsite	Granular fill*2	1982	Becker drilling and test pitting	Proposed source for concrete aggregate granular fill. Damsite area

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*1 During 1979 a general reconnaissance involving field mapping and hard excavated test pits were carried out over a wide area.

*2 Proposed sources for construction materials.

TABLE 11-1

MURPHY CREEK PROJECT

ESTIMATION OF CONSTRUCTION COSTS
($ million)

1.	Land and Rights	26.3
2.	Flowage	21.5
3.	Site Access and Clearing	5.7
4.	Cofferdams	16.7
5.	Approach Channel	18.4
6.	Discharge Channel	24.4
7.	Seepage Cut Off	29.9
8.	Embankment Dam	1.8
9.	Gravity Wall	2.8
10.	Spillway - Civil	27.5
	- Mechanical	10.5
11.	Powerhouse - Civil	110.7
	- Mechanical	112.2
	- Electrical	78.9
12.	Construction Services	18.6
	Direct Cost	505.9
13.	Contingencies	57.0
14.	Engineering, Final Design & Construction	50.2
15.	Hearing Expenses	2.0
16.	Mitigation, Compensation & Enhancement, not included above	9.3
17.	Construction Insurance and Bonds	5.4
18.	Engineering Studies	10.2
	Total Generation Project Construction Cost	640.0

Notes:

1. Costs are based on October 1982 prices.

2. Contingencies:

   - 10% on turbines and miscellaneous mechanical equipment.
   - 7.5% on all other equipment costs.
   - 12.5% on all other direct costs except for the seepage cut-off to which a 20% contingency has been applied.
   - 15.0% on miscellaneous items.

3. Engineering - Final Design and Construction: 9% on Direct Construction Cost plus contingencies.

TABLE 11-2

MURPHY CREEK PROJECT

ESTIMATE OF CONSTRUCTION EXPENDITURES BY FISCAL YEARS
($ Millions)

Units	Total Construction Cost	Prior to 83	Fiscal Year											
			83/84	84/85	85/86	86/87	87/88	88/89	89/90	90/91	91/92	92/93	93/94	94/95
1	495.0	6.0	2.6	4.3	3.4	6.2	13.5	41.1	47.9	127.9	148.4	78.6	10.5	4.6
2	36.3								1.5	5.8	7.2	19.6	2.2	
3	36.2								0.4	4.7	6.2	16.9	8.0	
4	36.2									3.1	6.2	11.8	14.6	0.5
5	36.3	–	–	–	–	–	–	–	—	1.4	5.8	7.2	19.8	2.1
	640.0	6.0	2.6	4.3	3.4	6.2	13.5	41.1	49.8	142.9	173.8	134.1	55.1	7.2

Unit In-Service Schedule

#1 Jul 93
#2 Oct 93
#3 Feb 94

#4 Jun 94
#5 Oct 94

Notes:

1. Costs are based on October 1982 prices.
2. Interest during construction, inflation and corporate overhead are not included.
3. Fiscal Year - 1 April to 31 March.
4. Expenditures based on construction schedule (Fig. 10-1) and on construction cost estimate (Table 11-1).
5. Common costs are included with Unit 1.

TABLE 11-3

MURPHY CREEK PROJECT

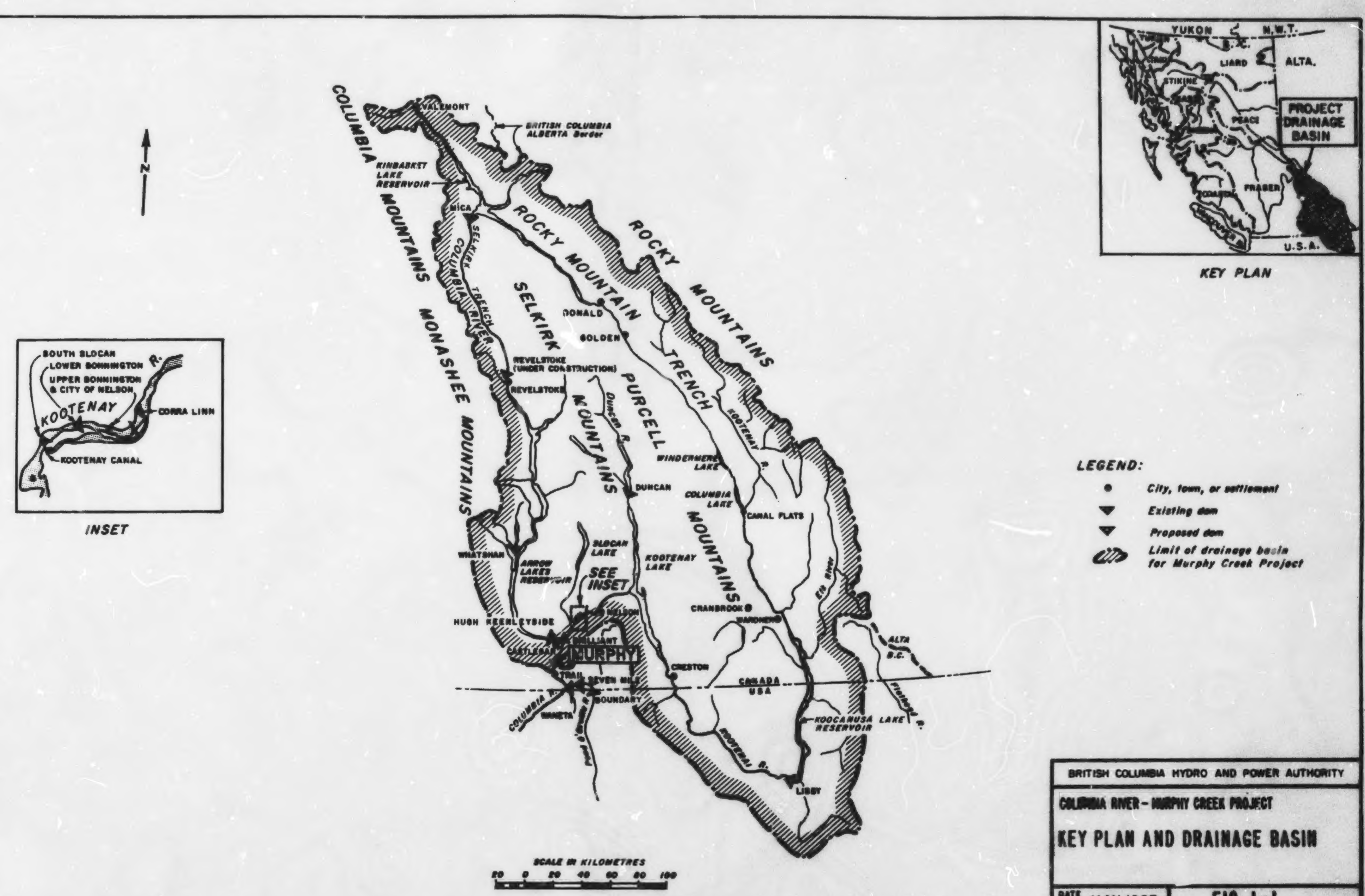
ESTIMATE OF LABOUR FORCE BY FISCAL YEARS

	Man-Years by Fiscal Year								
	87/88	88/89	89/90	90/91	91/92	92/93	93/94	94/95	Total Man-Years
Trade Union									
Tunnel and Rock	2	50	50	188	220	118	35	7	670
IBEW	4	10	10	25	33	85	75	13	255
Carpenters	-	10	13	170	180	70	15	2	460
Machinists	-	-	-	15	40	74	55	6	190
Operating Engr.	1	30	49	100	150	80	25	5	440
Boilermakers	-	-	-	-	4	15	15	1	35
OTEU	-	34	44	90	105	90	55	12	430
Culinary Workers	-	3	15	30	35	30	15	2	130
Teamsters	1	25	30	60	65	48	20	6	255
Cement Masons	-	-	-	6	16	14	4	-	40
Pipefitters	-	5	10	20	25	40	34	6	140
Ironworkers	-	5	13	50	55	40	15	2	180
Misc. Trades	-	4	4	5	10	25	15	2	65
Non-Union									
Supervisory	1	12	22	35	42	31	22	5	170
BCH staff	1	12	15	26	30	30	20	6	140
Total Man-Years	10	200	275	820	1010	790	420	75	3600
Peak Manpower	50	300	400	1000	1160	1000	600	250	

Notes: 1. Fiscal year - 1 April to 31 March.

2. Labour force estimate based on estimate of capital expenditures (Tables 11-2).

FIGURES

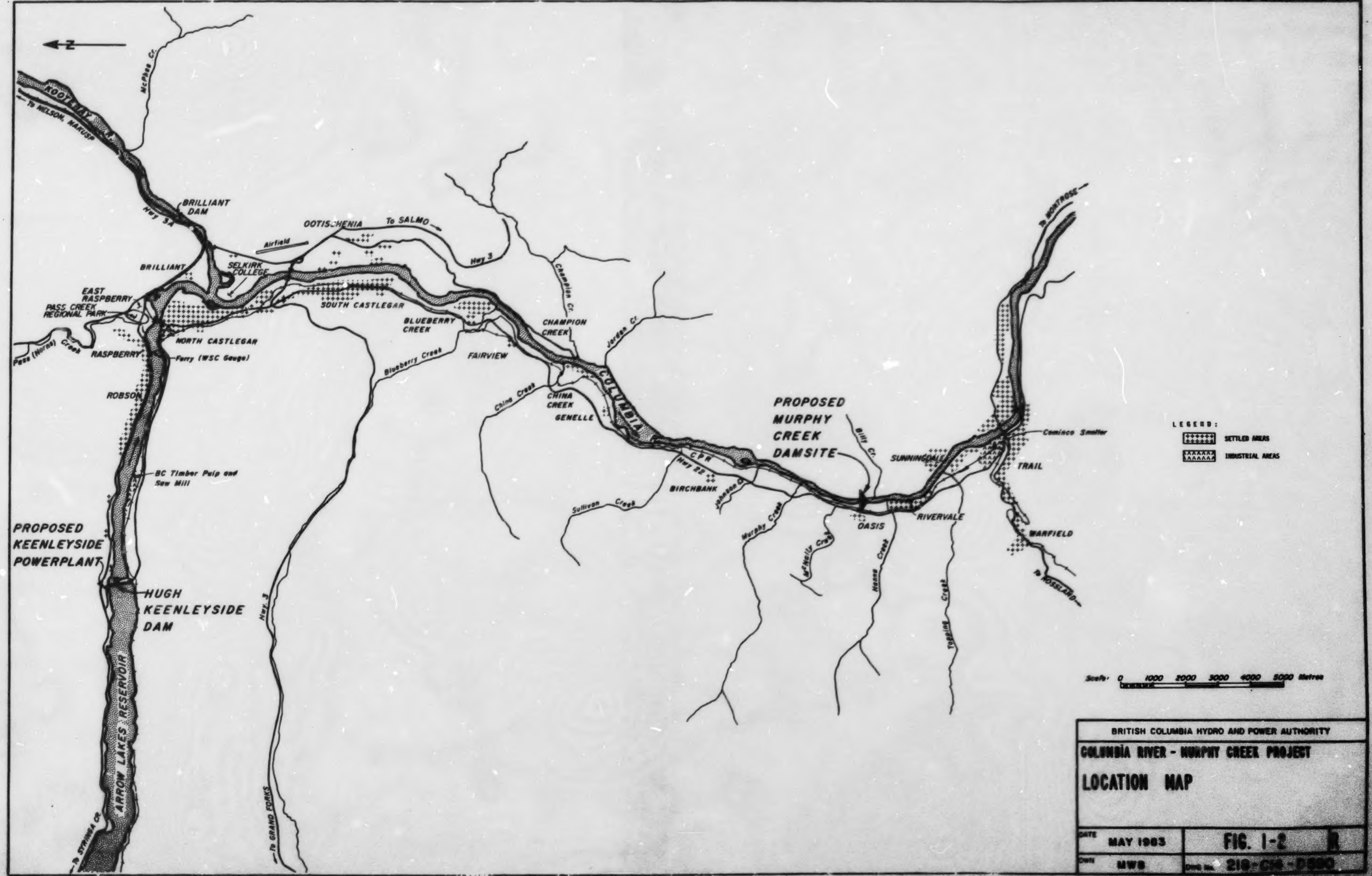

N
VALEMONT
BRITISH COLUMBIA
ALBERTA Border
COLUMBIA MOUNTAINS
KINBASKET LAKE RESERVOIR
MICA
ROCKY MOUNTAIN TRENCH
ROCKY MOUNTAINS
SELKIRK
COLUMBIA RIVER
TRENCH
SELKIRK MOUNTAINS
MONASHEE MOUNTAINS
DONALD
GOLDEN
REVELSTOKE (UNDER CONSTRUCTION)
REVELSTOKE
PURCELL MOUNTAINS
Duncan R.
KOOTENAY R.
WINDERMERE LAKE
DUNCAN
COLUMBIA LAKE
CANAL FLATS
SLOCAN LAKE
KOOTENAY LAKE
WHATSHAN
ARROW LAKES RESERVOIR
SEE INSET
NELSON
HUGH KEENLEYSIDE
BRILLIANT
CASTLEGAR
MURPHY
CRANBROOK
WARDNER
Elk River
ALTA
B.C.
CRESTON
CANADA
USA
TRAIL
SEVEN MILE
BOUNDARY
WANETA
COLUMBIA R.
Pend d'Oreille R.
KOOTENAI R.
KOOCANUSA LAKE RESERVOIR
Flathead R.
LIBBY
SCALE IN KILOMETRES
20 0 20 40 60 80 100
SOUTH SLOCAN
LOWER BONNINGTON
UPPER BONNINGTON & CITY OF NELSON
KOOTENAY R.
CORRA LINN
KOOTENAY CANAL
INSET
YUKON
N.W.T.
B.C.
LIARD
ALTA.
STIKINE
PEACE
PROJECT DRAINAGE BASIN
FRASER
U.S.A.
KEY PLAN
LEGEND:
● City, town, or settlement
▼ Existing dam
▽ Proposed dam
▨ Limit of drainage basin for Murphy Creek Project

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER – MURPHY CREEK PROJECT

KEY PLAN AND DRAINAGE BASIN

DATE MAY 1983	FIG 1-1
DWN MWB	218-C14-B589


N
KOOTENAY R.
To NELSON, NAKUSP
McPhee Cr.
Hwy 3A
BRILLIANT DAM
BRILLIANT
OOTISCHENIA
To SALMO
Airfield
Hwy 3
SELKIRK COLLEGE
EAST RASPBERRY
PASS CREEK REGIONAL PARK
Pass (Norns) Creek
RASPBERRY
NORTH CASTLEGAR
Ferry (WSC Gauge)
SOUTH CASTLEGAR
BLUEBERRY CREEK
Blueberry Creek
FAIRVIEW
Champion Cr.
CHAMPION CREEK
Jordan Cr.
ROBSON
China Creek
CHINA CREEK
GENELLE
COLUMBIA
CPR
Hwy 22
BIRCHBANK
Johnson Cr.
Sullivan Creek
PROPOSED MURPHY CREEK DAMSITE
Billy Cr.
Murphy Creek
McNally Creek
OASIS
Hanna Creek
SUNNINGDALE
RIVERVALE
Topping Creek
Cominco Smelter
TRAIL
WARFIELD
To ROSSLAND
To MONTROSE
BC Timber Pulp and Saw Mill
PROPOSED KEENLEYSIDE POWERPLANT
HUGH KEENLEYSIDE DAM
ARROW LAKES RESERVOIR
To SYRINGA Cr.
Hwy 3
To GRAND FORKS
LEGEND:
SETTLED AREAS
INDUSTRIAL AREAS
Scale: 0 1000 2000 3000 4000 5000 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

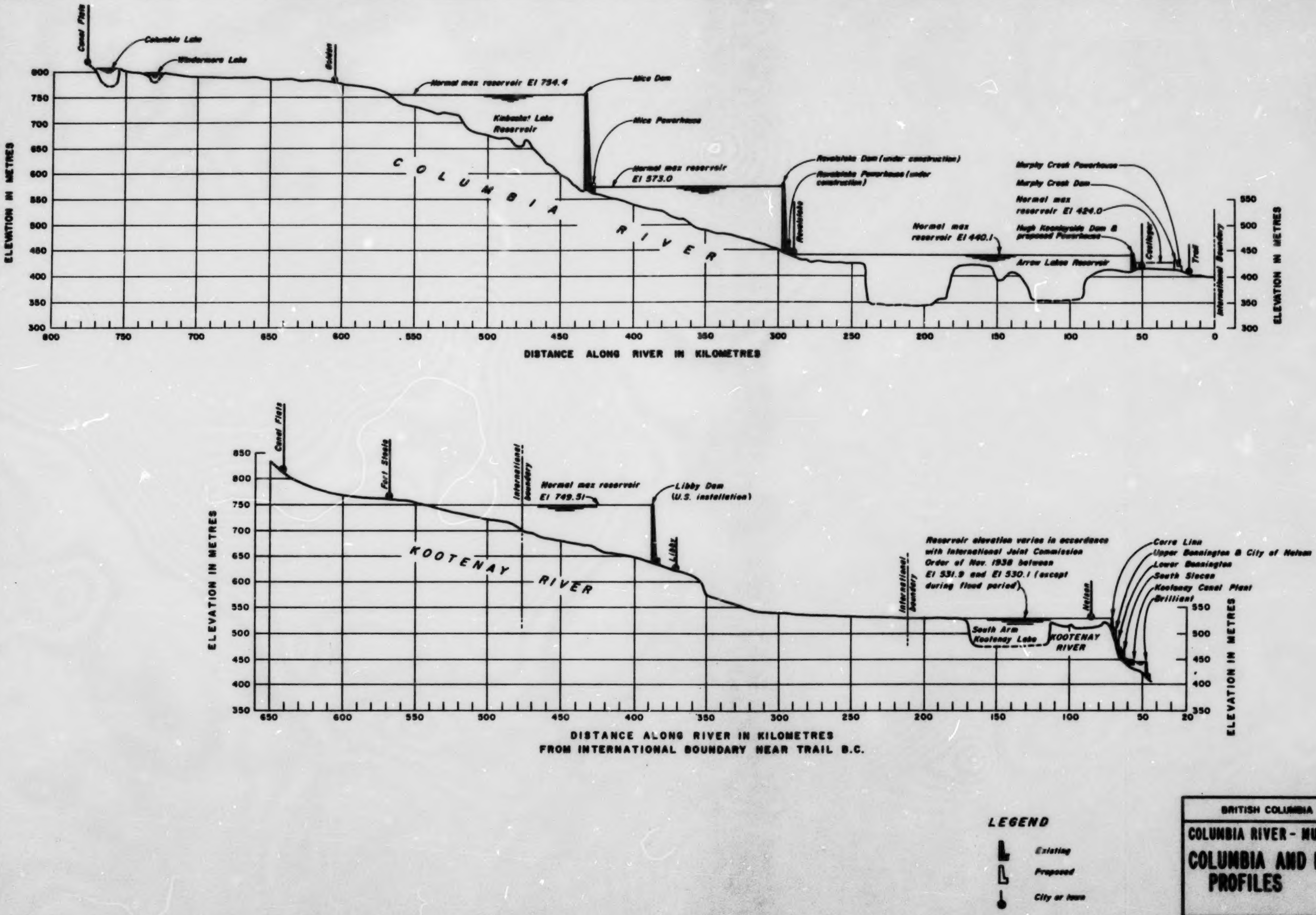
LOCATION MAP

DATE MAY 1983

FIG. 1-2

DWN MWB

DWG No. 218-C14-D990

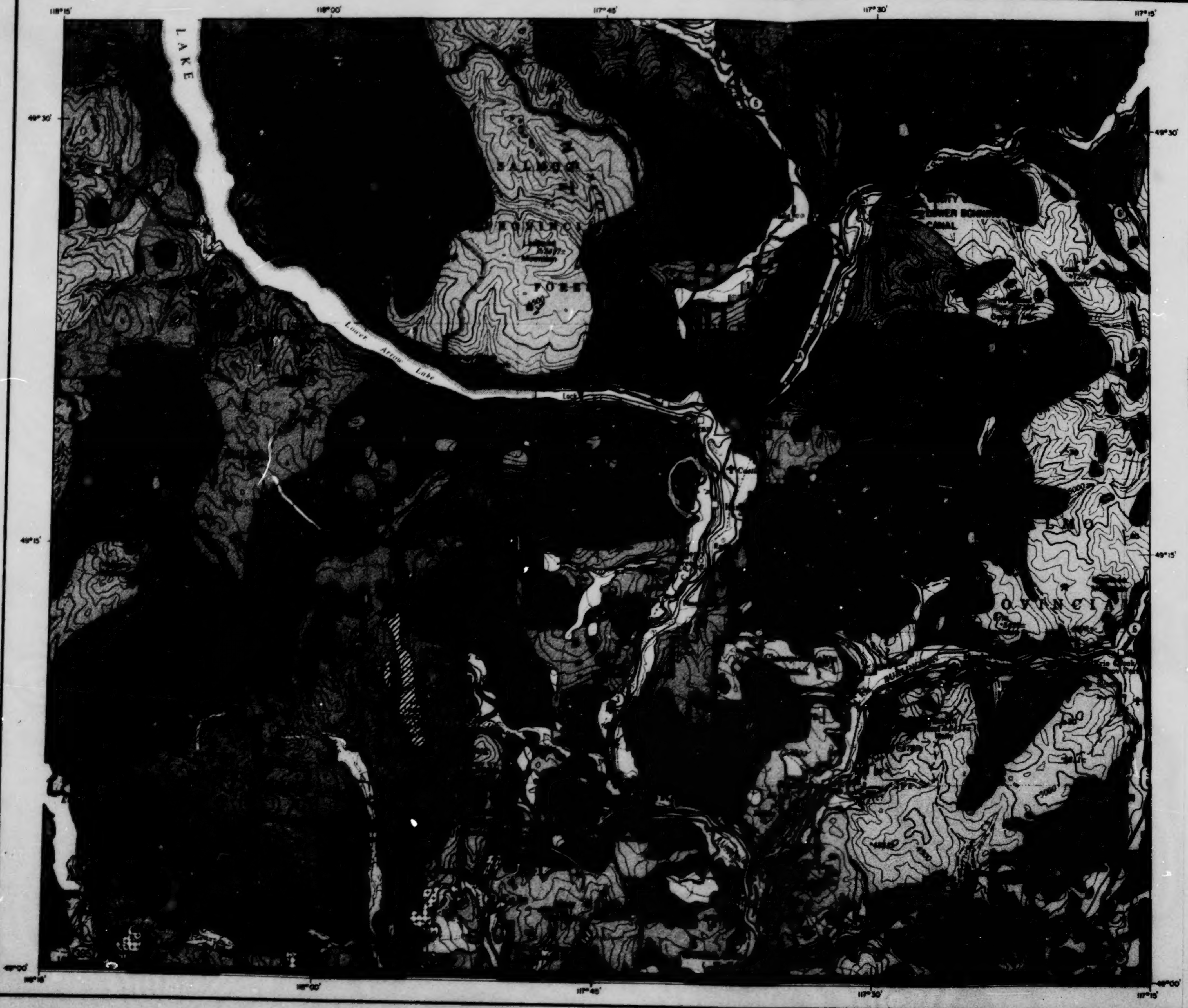

Canal Flats
Columbia Lake
Windermere Lake
Golden
Normal max reservoir El 754.4
Mica Dam
Kinbasket Lake Reservoir
Mica Powerhouse
Normal max reservoir El 573.0
COLUMBIA RIVER
Revelstoke Dam (under construction)
Revelstoke Powerhouse (under construction)
Revelstoke
Normal max reservoir El 440.1
Arrow Lakes Reservoir
Murphy Creek Powerhouse
Murphy Creek Dam
Normal max reservoir El 424.0
Hugh Keenleyside Dam & proposed Powerhouse
Castlegar
Trail
International Boundary
ELEVATION IN METRES
900
850
800
750
700
650
600
550
500
450
400
350
300
800
750
700
650
600
550
500
450
400
350
300
250
200
150
100
50
0
DISTANCE ALONG RIVER IN KILOMETRES
Canal Flats
Fort Steele
International boundary
Normal max reservoir El 749.51
Libby Dam (U.S. installation)
Libby
KOOTENAY RIVER
International boundary
Reservoir elevation varies in accordance with International Joint Commission Order of Nov. 1938 between El 531.9 and El 530.1 (except during flood period)
Nelson
South Arm Kootenay Lake
KOOTENAY RIVER
Corra Linn
Upper Bonnington & City of Nelson
Lower Bonnington
South Slocan
Kootenay Canal Plant
Brilliant
ELEVATION IN METRES
850
800
750
700
650
600
550
500
450
400
350
650
600
550
500
450
400
350
300
250
200
150
100
50
20
DISTANCE ALONG RIVER IN KILOMETRES
FROM INTERNATIONAL BOUNDARY NEAR TRAIL B.C.
LEGEND
Existing
Proposed
City or town

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

COLUMBIA AND KOOTENAY RIVER PROFILES

DATE MAY 1983	FIG. 2-1	R
[illegible] MWB	[illegible] 216 - C14 - D[illegible]	


118°15'
118°00'
117°45'
117°30'
117°15'
49°30'
49°15'
49°00'
LAKE
Lower Arrow Lake
SALMO

LEGEND:

CENOZOIC

QUATERNARY
PLEISTOCENE AND RECENT
▭ Till, gravel, sand and silt

TERTIARY
LOWER TERTIARY
■ CORYELL PLUTONIC ROCKS (mostly syenite) and SHEPFARD INTRUSIONS (leucocratic granite)

EOCENE (?)
▨ Agglomerate and lava flows

CRETACEOUS OR EARLY TERTIARY
UPPER CRETACEOUS (?)
▧ SOPHIE MOUNTAIN FORMATION (conglomerate)
LOWER CRETACEOUS (?)
▭ VALHALLA INTRUSIONS (granite)

MESOZOIC

JURASSIC OR CRETACEOUS
UPPER JURASSIC OR LOWER CRETACEOUS
■ NELSON PLUTONIC ROCKS (granitic rocks), including ROSSLAND monzonite and minor quartz - feldspar porphyry
▦ Ultrabasic rocks, serpentinite

JURASSIC
LOWER, MIDDLE AND (?) UPPER JURASSIC
▭ ROSSLAND GROUP (volcanic rocks, siltstone and shale) including HALL FORMATION AND ELISE FORMATION
LOWER JURASSIC AND (?) OLDER
■ ARCHIBALD FORMATION (argillite & siltstone) and SINEMURIAN BEDS (argillite and quartzite) including minor YMIR GROUP (argillite)

PALAEOZOIC

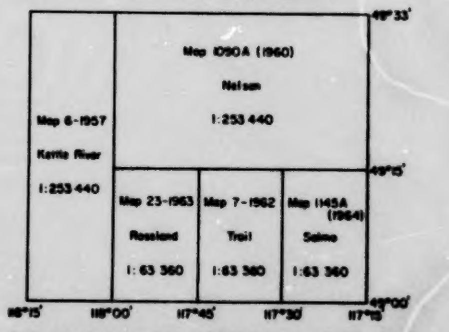
CARBONIFEROUS
■ MOUNT ROBERTS FORMATION (slate, limestone and quartzite)

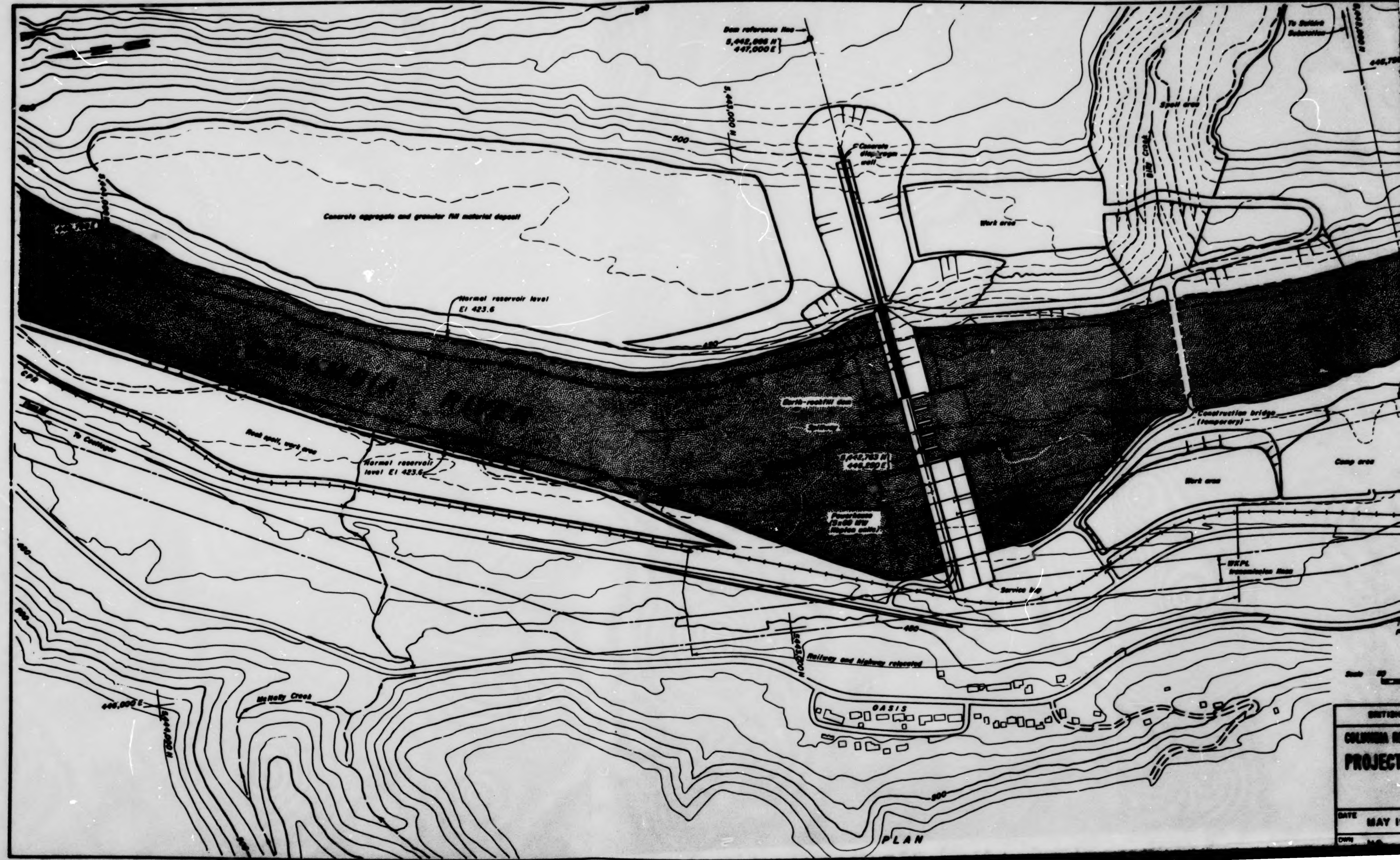
ORDOVICIAN AND CAMBRIAN
■ ACTIVE, NELWAY, LAIB, RENO AND QUARTZITE RANGE FORMATIONS (argillite, limestone, dolomite and quartzite)

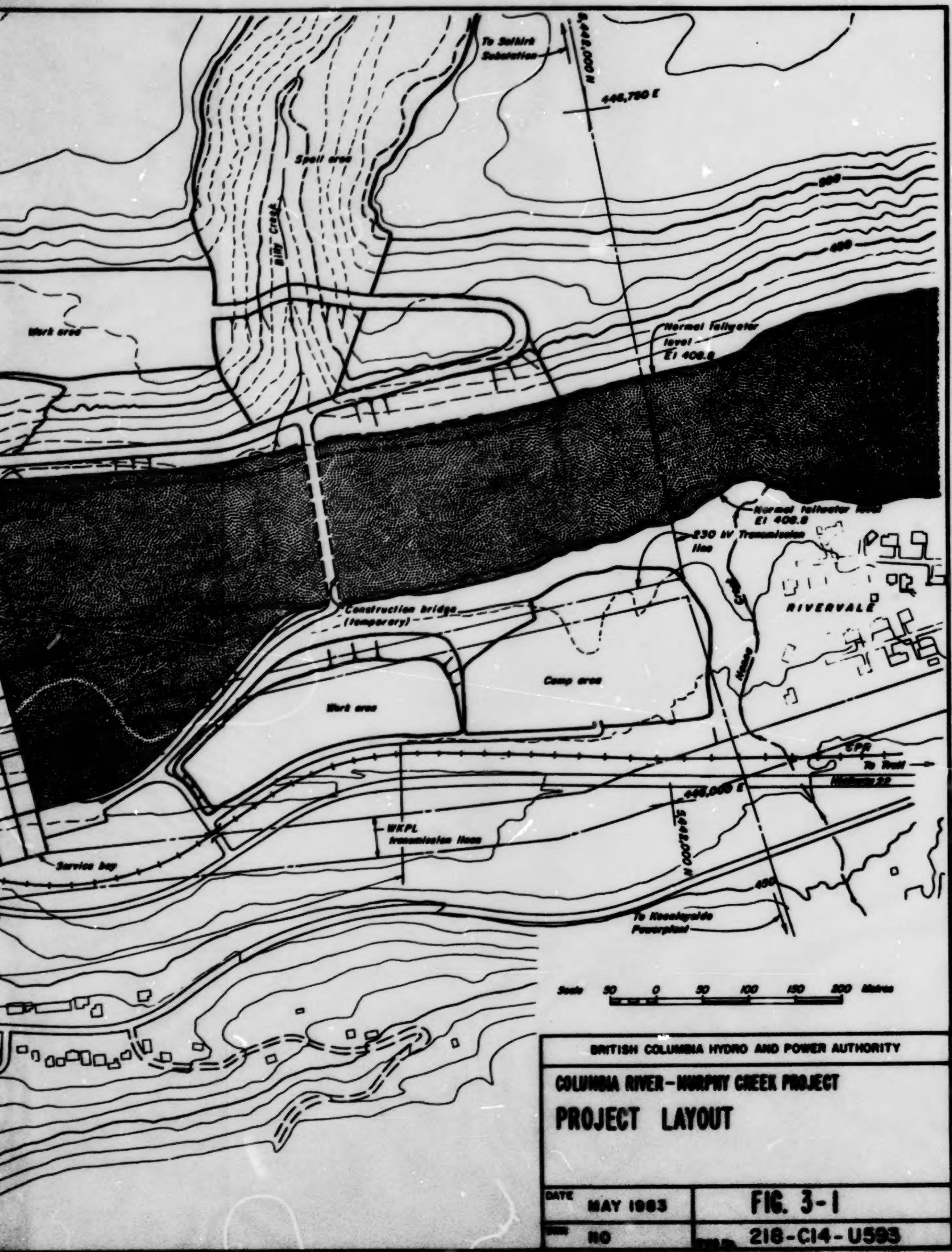
AGE UNKNOWN
▭ Mostly gneiss, including some argillite (especially along Pend D'Oreille River)

✕ 60 Strike and Dip of Bedding (Vertical, inclined)

✕ 40 Strike and Dip of Gneissosity (Vertical, inclined)

~~~ Fault (approximate or assumed)

NOTES:

1. This map is based on the following maps by H.W. Little of the Geological Survey of Canada:


Map 6-1957
Kettle River
1:253 440
Map 1090A (1960)
Nelson
1:253 440
Map 23-1963
Rossland
1:63 360
Map 7-1962
Trail
1:63 360
Map 1145A (1964)
Salmo
1:63 360
49°33'
49°15'
49°00'
118°15'
118°00'
117°45'
117°30'
117°15'

2. Topography is taken from National Topographic System maps 82E and 82F (1973 edition, elevations in feet)

Scale 2.5 0 2.5 5 7.5 10 Kilometres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

REGIONAL GEOLOGY

DATE	MAY 1983	FIG. 2-2	
DWN	GC	DWG No.	218-C14-D592

REPORT No. H1617
~~~


Dam reference line
5,442,866 N
447,000 E
5,443,000 N
500
Concrete diaphragm wall
To Selkirk Substation
Spoil area
Billy Creek
Work area
Concrete aggregate and granular fill material deposit
Normal reservoir level
El 423.6
COLUMBIA RIVER
CPR
To Castlegar
Rock spoil, work area
Normal reservoir level El 423.6
Earth-rockfill dam
Spillway
5,442,763 N
446,280 E
Powerhouse
(3x60 MW
Kaplan units)
Construction bridge
(temporary)
Camp area
Work area
WKPL
transmission lines
Service bay
450
Railway and highway relocated
OASIS
McNally Creek
446,000 E
5,444,000 N
PLAN
Scale
COLUMBIA
PROJECT
DATE
MAY
DWN


To Selkirk Substation
5,442,000 N
446,750 E
Spoil area
Billy Creek
500
450
Work area
Normal tailwater level El 406.8
Normal tailwater level El 406.8
230 kV Transmission line
Creek
Hanna
RIVERVALE
Construction bridge (temporary)
Camp area
Work area
CPR
To Trail
Highway 22
446,000 E
5,442,000 N
WKPL transmission lines
Service bay
450
To Kootenayside Powerplant
Scale 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER–MURPHY CREEK PROJECT

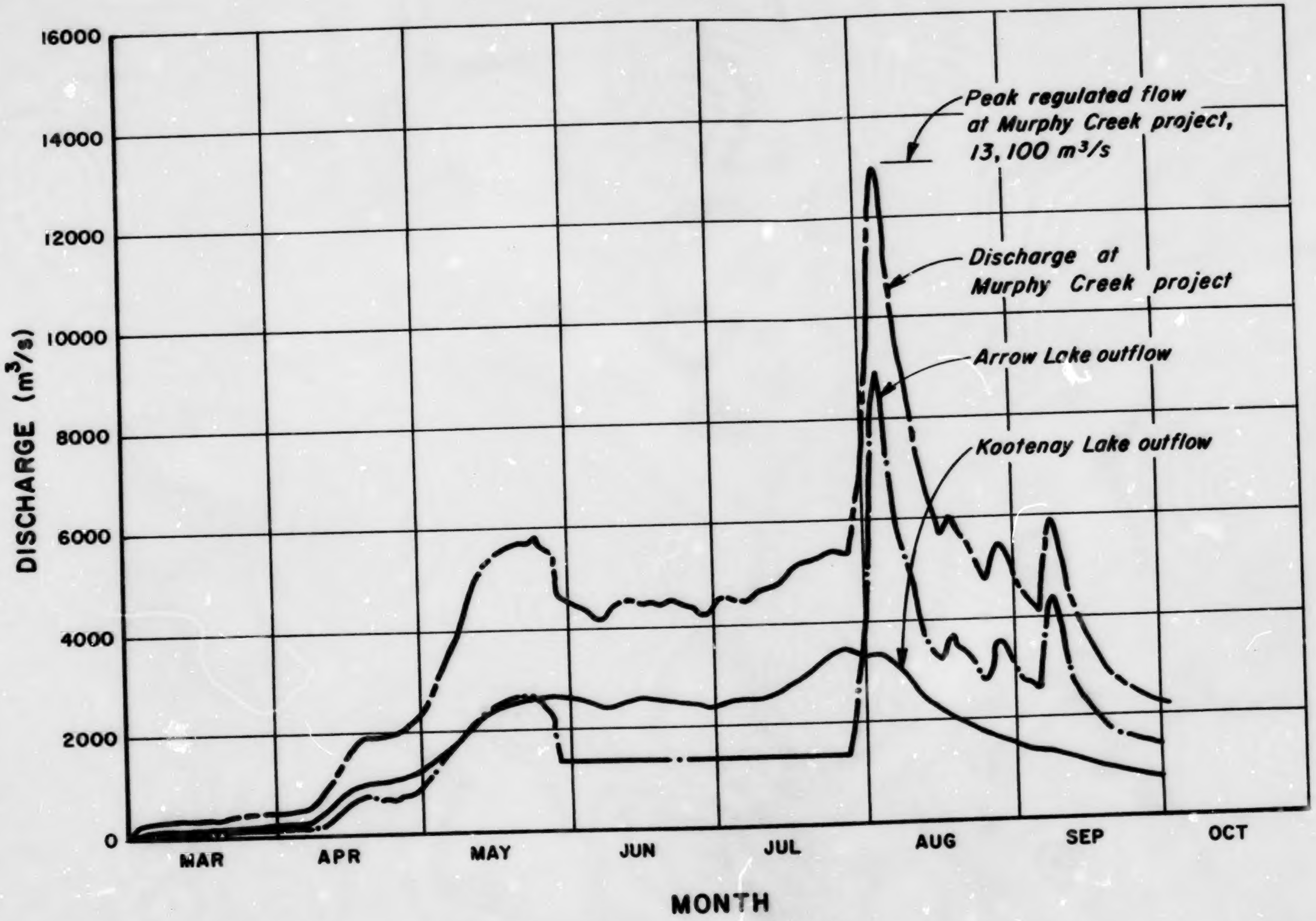
PROJECT LAYOUT

DATE MAY 1983

FIG. 3-1

HO

218-C14-U593



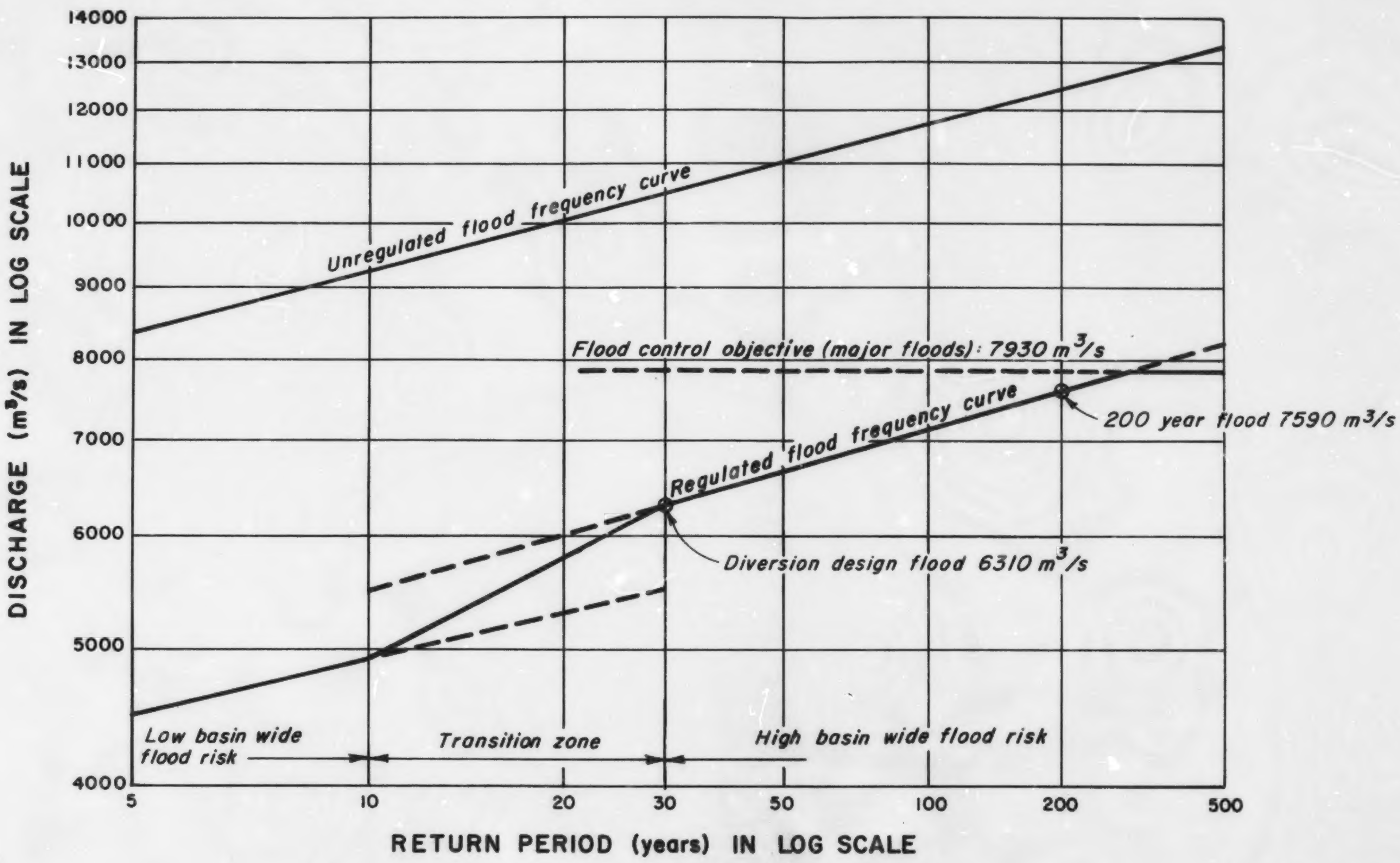
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

PROBABLE MAXIMUN FLOOD HYDROGRAPH

DATE MAY 1983	FIG. 4-1
DWN EC jr	DWG. No. 218-C14-B594

REPORT No. H1617

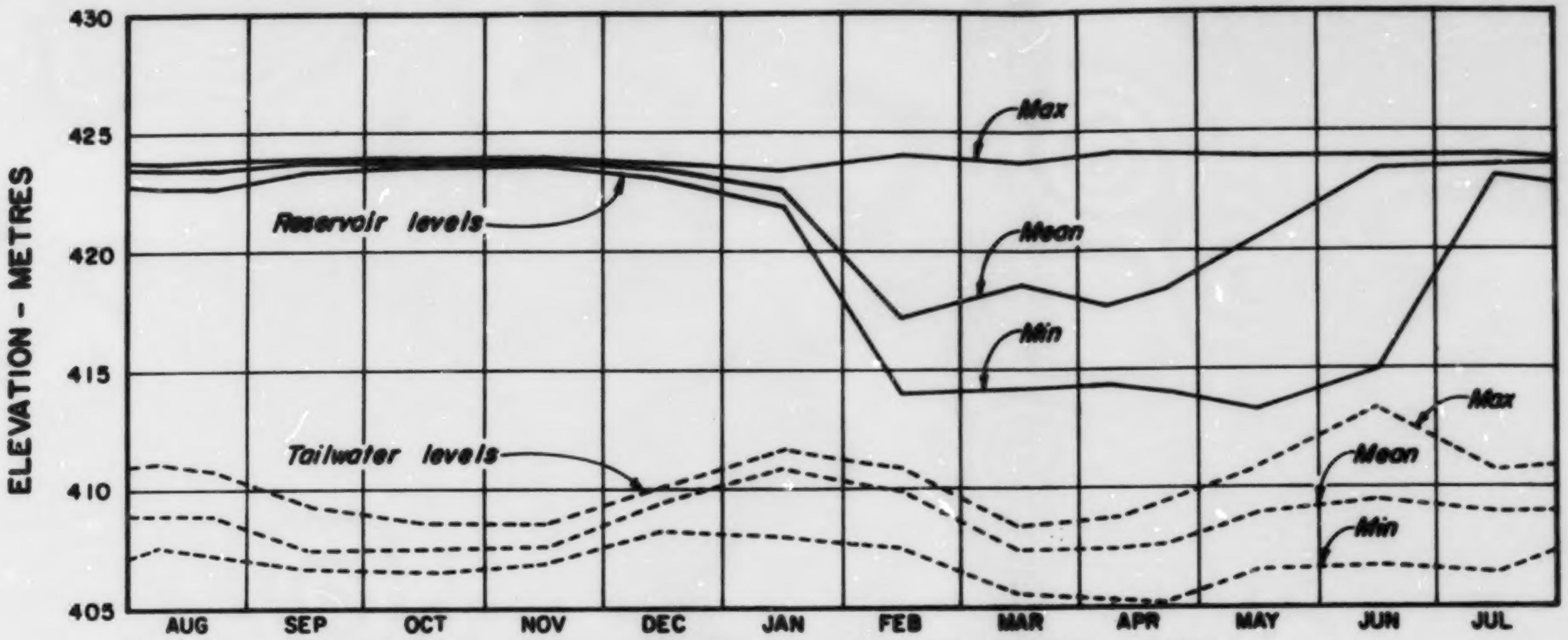


BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

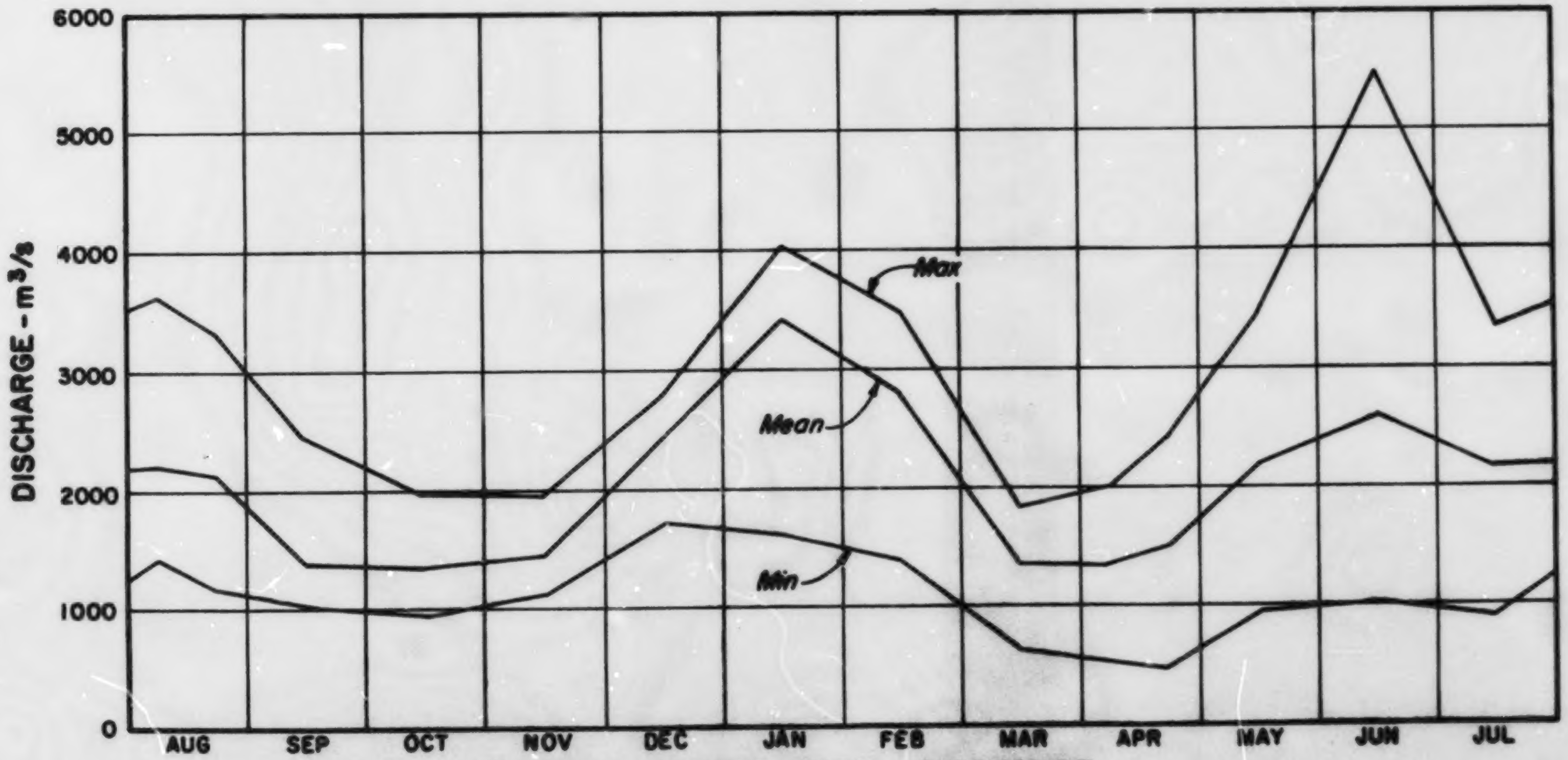
COLUMBIA RIVER - MURPHY CREEK PROJECT

REGULATED FLOOD FREQUENCY CURVE

DATE MAY 1983	FIG. 4-2
DWN ECjr	DWG. No. 218-C14-B595



MURPHY CREEK RESERVOIR AND TAILWATER LEVEL



MURPHY CREEK TOTAL DISCHARGE

NOTE:

RESERVOIR LEVELS SHOWN ARE AT THE DAM AND ARE BASED ON A MAXIMUM OPERATING LEVEL OF EL. 424 m AT CASTLEGAR FERRY WATER LEVEL GAUGE.

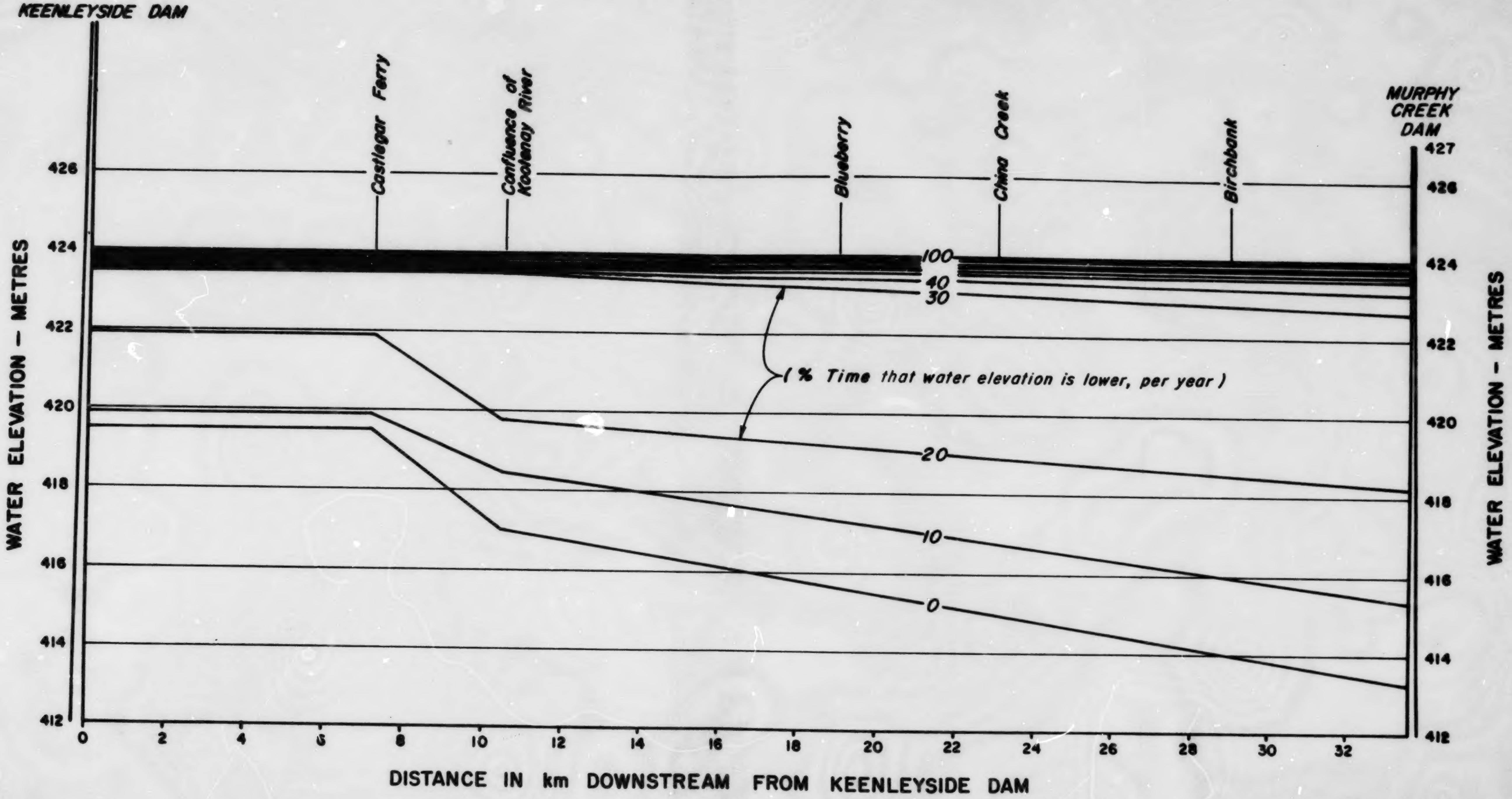
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER—MURPHY CREEK PROJECT

WATER LEVELS AND
DISCHARGE HYDROGRAPHS -
REGIME I

DATE MAY 1983
FIG. 4-3
DRN FL
DWG. No 218-C14-B596

REPORT No [illegible]



NOTES:

1. RESERVOIR LEVELS SHOWN ARE BASED ON A MAXIMUM OPERATING LEVEL OF EL. 424 m AT CASTLEGAR FERRY WATER LEVEL GAUGE.
2. RESERVOIR LEVEL DATA IS BASED ON 35 YEAR MONTHLY FLOW SIMULATION.

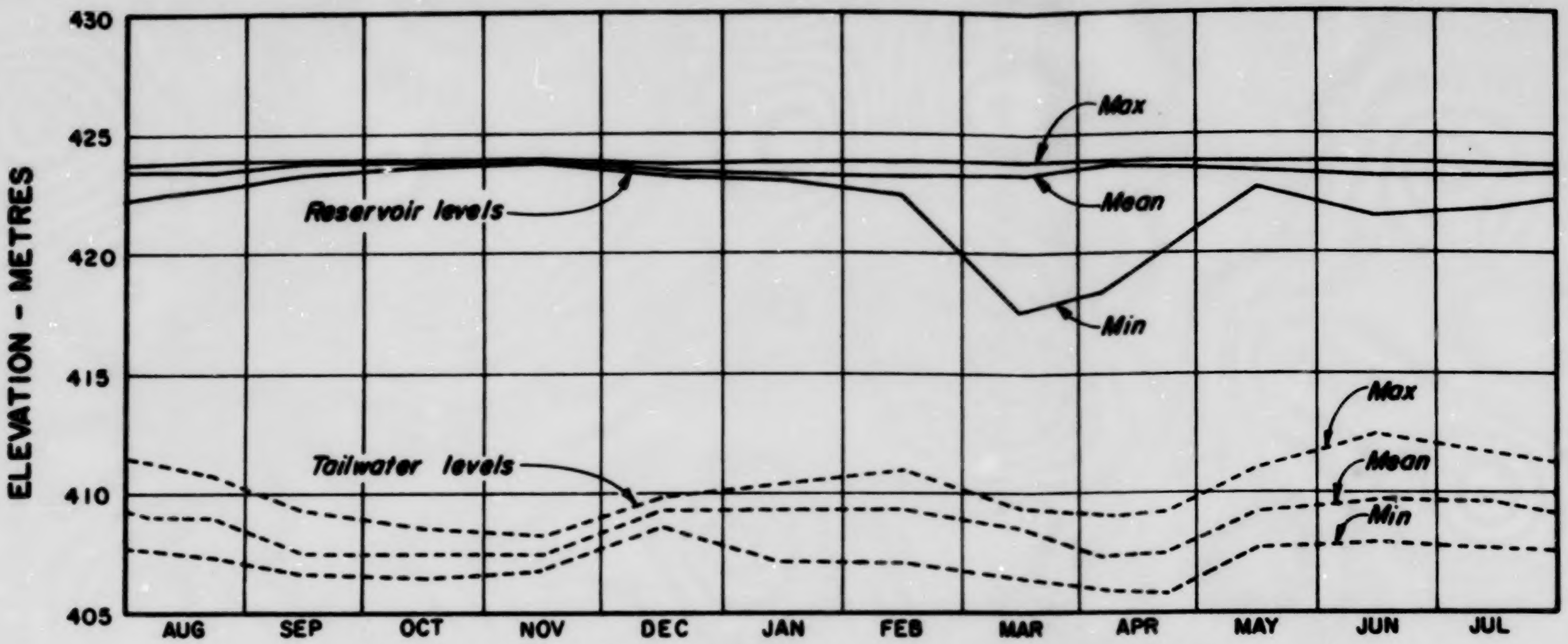
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

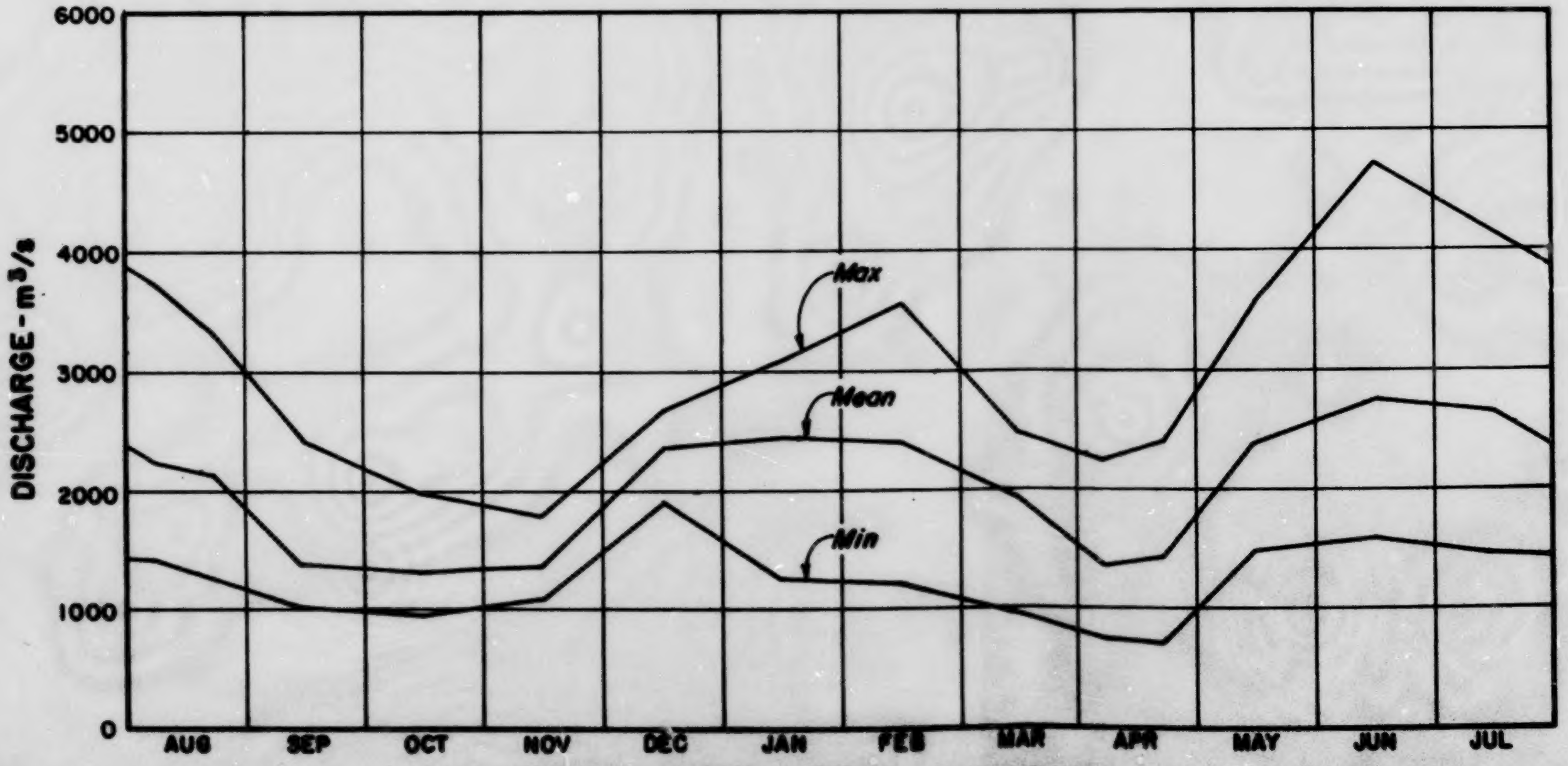
RESERVOIR LEVELS - REGIME I

DATE MAY 1983	FIG. 4-4
DWN FL	DWG. No 218-C14-B597

REPORT No H1617



MURPHY CREEK RESERVOIR AND TAILWATER LEVELS



MURPHY CREEK TOTAL DISCHARGE

NOTE:

RESERVOIR LEVELS SHOWN ARE AT THE DAM AND ARE BASED ON A MAXIMUM OPERATING LEVEL OF EL. 424 m AT CASTLEGAR FERRY WATER LEVEL GAUGE.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER – MURPHY CREEK PROJECT

WATER LEVELS AND DISCHARGE
HYDROGRAPHS -
REGINE 2

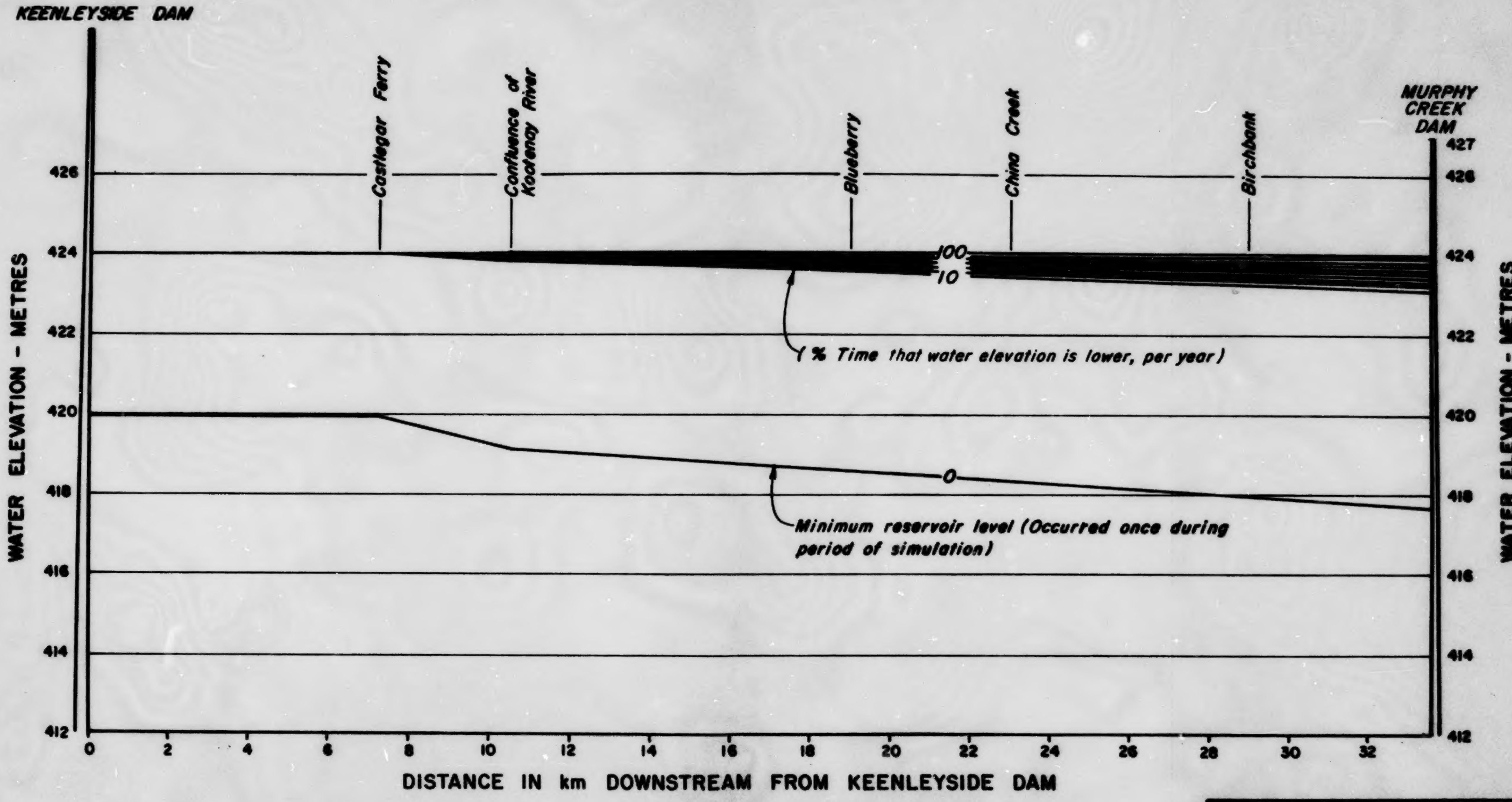
DATE MAY 1983

FIG. 4-5

FL

218-C14-8598

REPORT No. H1617



NOTES:

1. RESERVOIR LEVELS SHOWN ARE BASED ON A MAXIMUM OPERATING LEVEL OF EL. 424 m AT CASTLEGAR FERRY WATER LEVEL GAUGE.
2. RESERVOIR LEVEL DATA IS BASED ON 35 YEAR MONTHLY FLOW SIMULATION.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER – MURPHY CREEK PROJECT

RESERVOIR LEVELS - REGIME 2

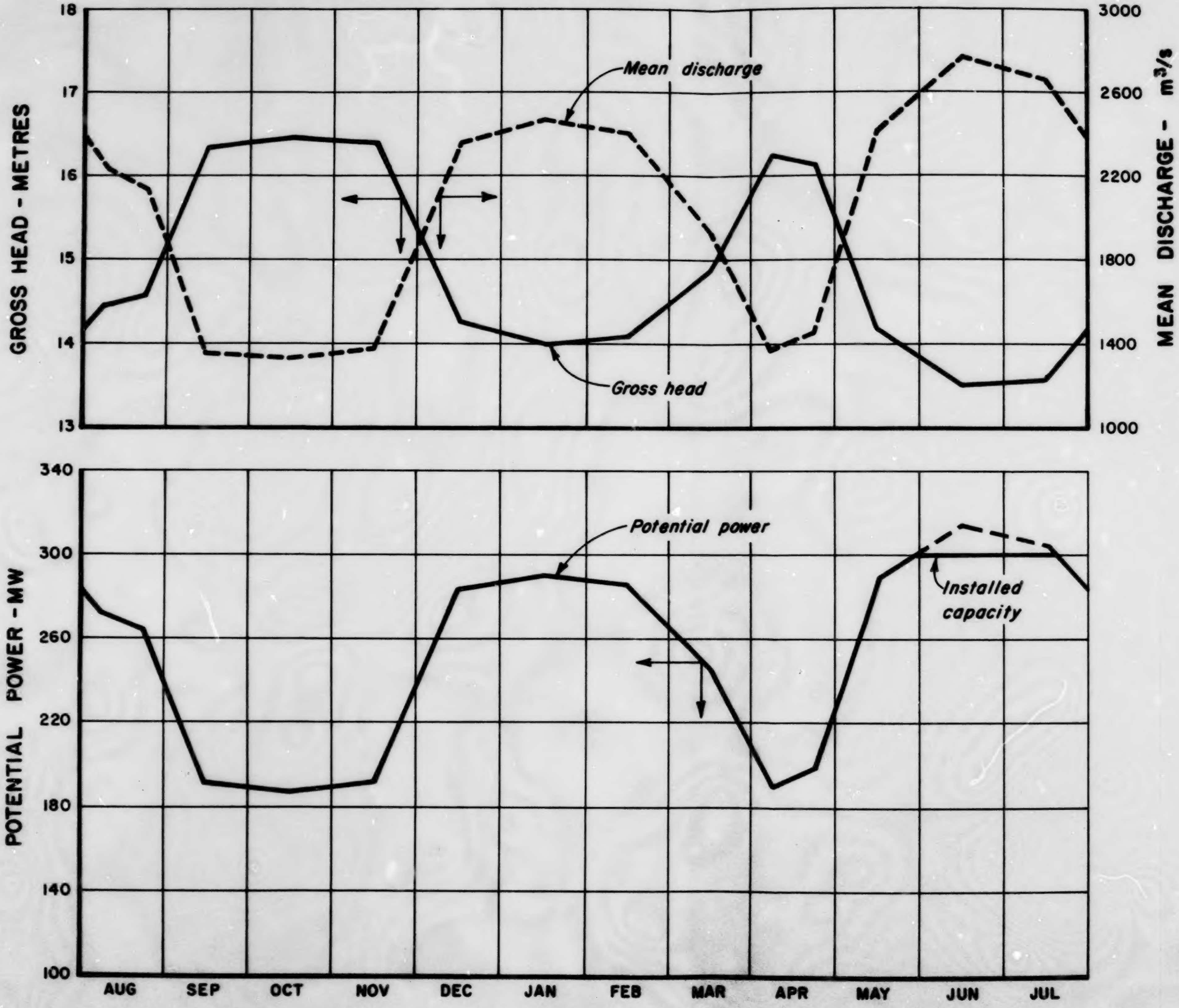
DATE MAY 1983

FIG. 4-6

DWN FL

DWG. No. 218-C14-B599

REPORT No. H1617



NOTES:

1. POTENTIAL POWER IS BASED ON 35 YEARS OF SIMULATED MEAN MONTHLY VALUES OF GROSS HEAD AND TOTAL DISCHARGE. AN AVERAGE HEAD LOSS OF 0.6 m AND AN OVERALL PLANT EFFICIENCY OF 0.90 HAVE BEEN ASSUMED.
2. GROSS HEAD VALUES ARE BASED ON MAXIMUM RESERVOIR OPERATING LEVEL OF EL. 424 m AT CASTLEGAR FERRY WATER LEVEL GAUGE.

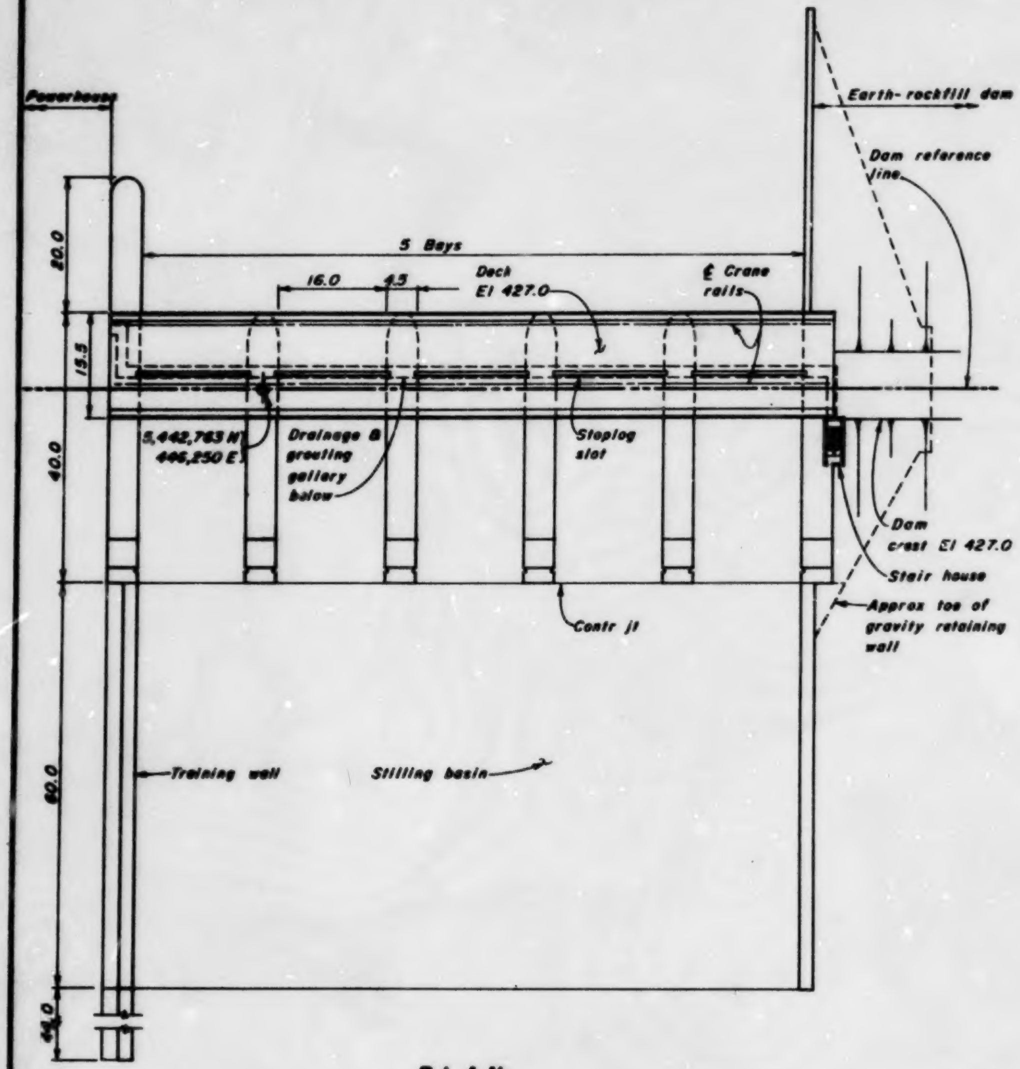
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

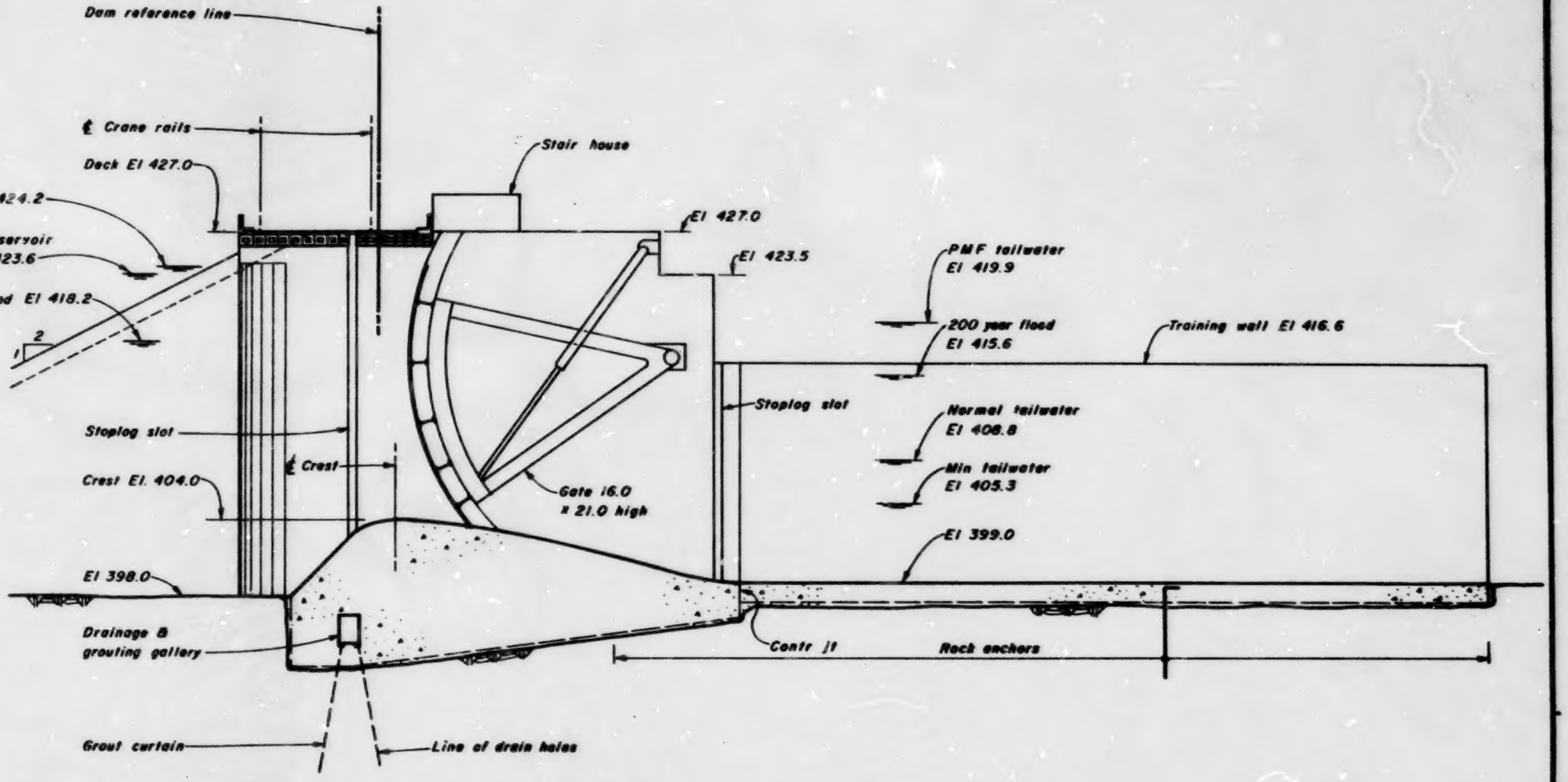
HEAD, DISCHARGE AND MONTHLY POWER POTENTIAL

DATE MAY 1983	FIG. 4-7
DWN MWB	DWG. No. 218-C14-B600

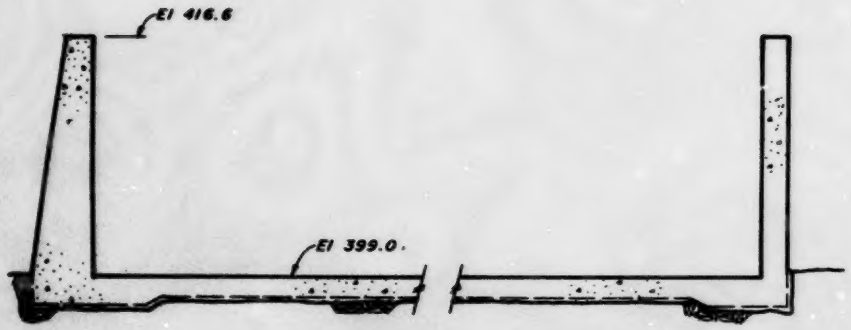
REPORT No. H1617


Powerhouse
Earth-rockfill dam
Dam reference line
5 Bays
16.0
4.5
Deck El 427.0
℄ Crane rails
20.0
15.5
5,442,763 N
446,250 E
Drainage & grouting gallery below
Stoplog slot
40.0
Dam crest El 427.0
Stair house
Contr jt
Approx toe of gravity retaining wall
Training wall
Stilling basin
60.0
44.0

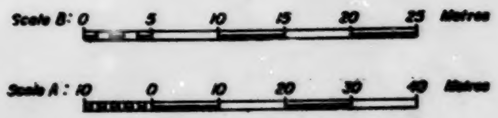
PLAN
Scale A


Dam reference line
℄ Crane rails
Deck El 427.0
Stair house
PMF El 424.2
Normal reservoir level El 423.6
200 yr. flood El 418.2
2
1
El 427.0
El 423.5
PMF tailwater El 419.9
200 year flood El 415.6
Training wall El 416.6
Stoplog slot
Stoplog slot
℄ Crest
Crest El. 404.0
Gate 16.0 x 21.0 high
Normal tailwater El 408.8
Min tailwater El 405.3
El 399.0
El 398.0
Drainage & grouting gallery
Contr jt
Rock anchors
Grout curtain
Line of drain holes

TYPICAL SPILLWAY SECTION
Scale B


El 416.6
El 399.0

SECTION THROUGH STILLING BASIN
Scale B


Scale B: 0 5 10 15 20 25 Metres
Scale A: 10 0 10 20 30 40 Metres

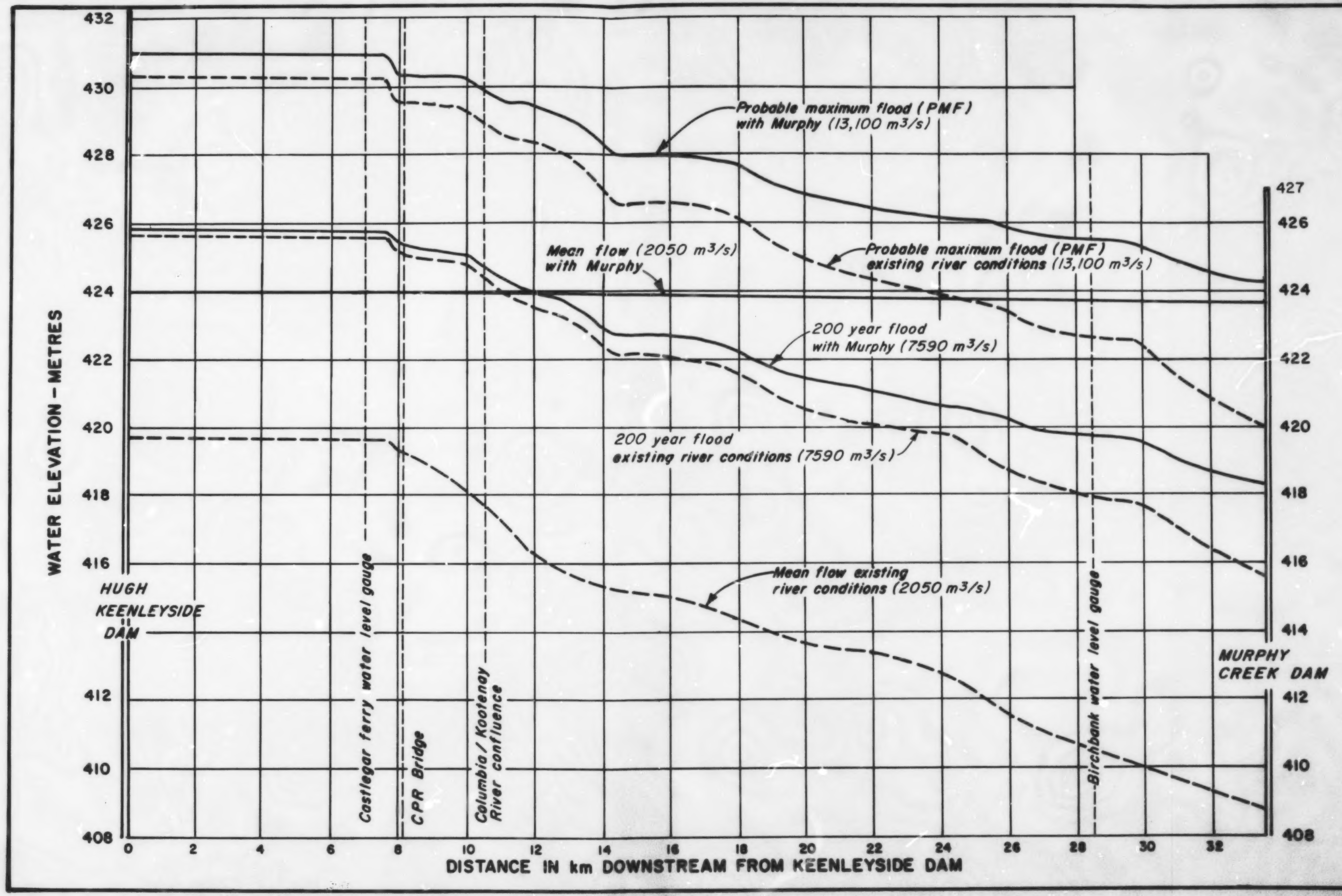
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

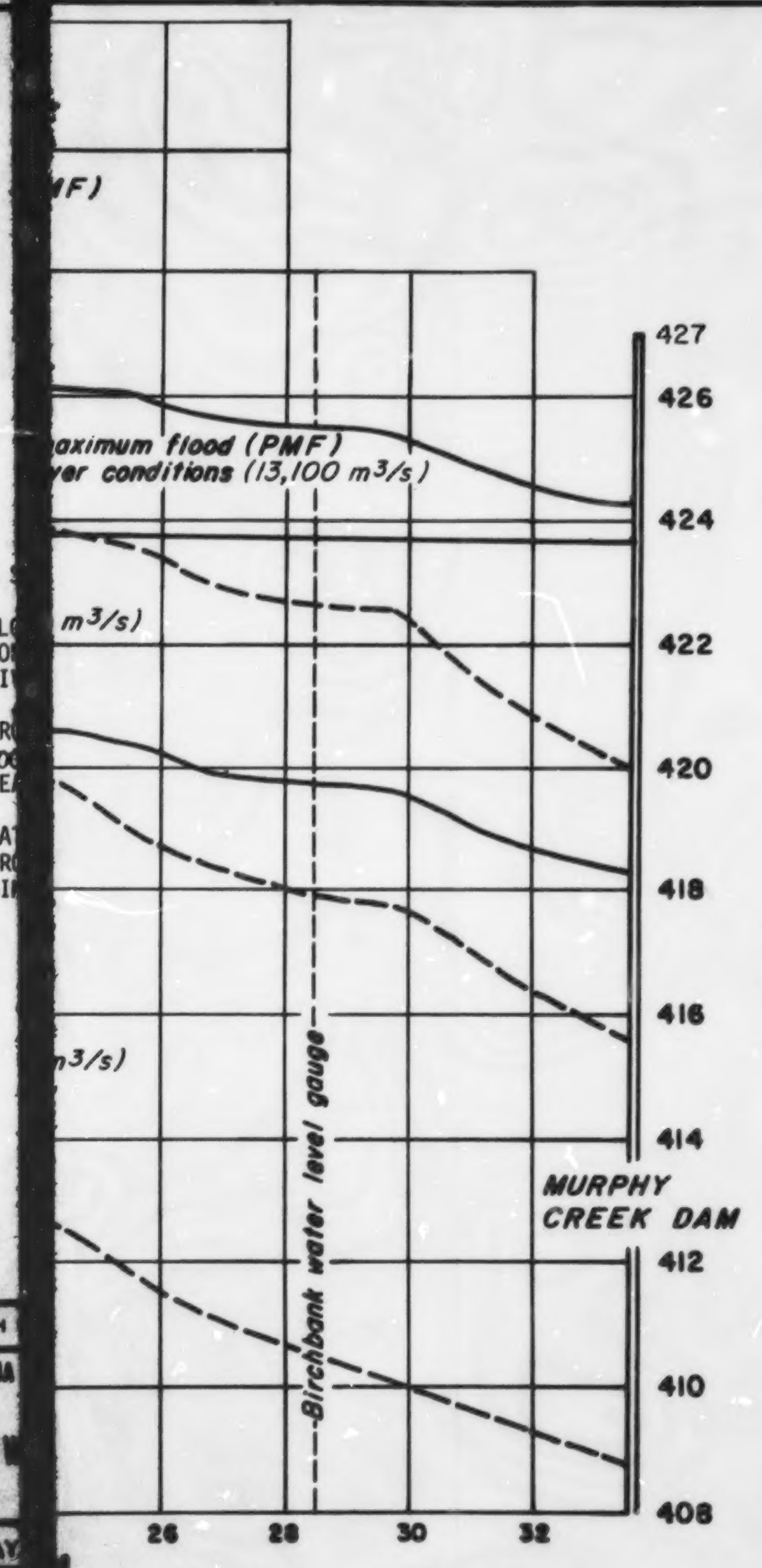
COLUMBIA RIVER - MURPHY CREEK PROJECT

SPILLWAY
GENERAL ARRANGEMENT

DATE MAY 1983	FIG.5-1		R
DWN MWB	DWG No. 218-C14-D630		

REPORT No. H1617





NOTES:

1. FLOW SPLITS FOR THE COLUMBIA/KOOTENAY RIVERS ARE:

PROBABLE MAXIMUM FLOOD	-69/31
200 YEAR FLOOD	-55/45
MEAN FLOW	-56/44

2. WATER ELEVATIONS WERE DERIVED FROM A COMPUTER BACKWATER SIMULATION.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

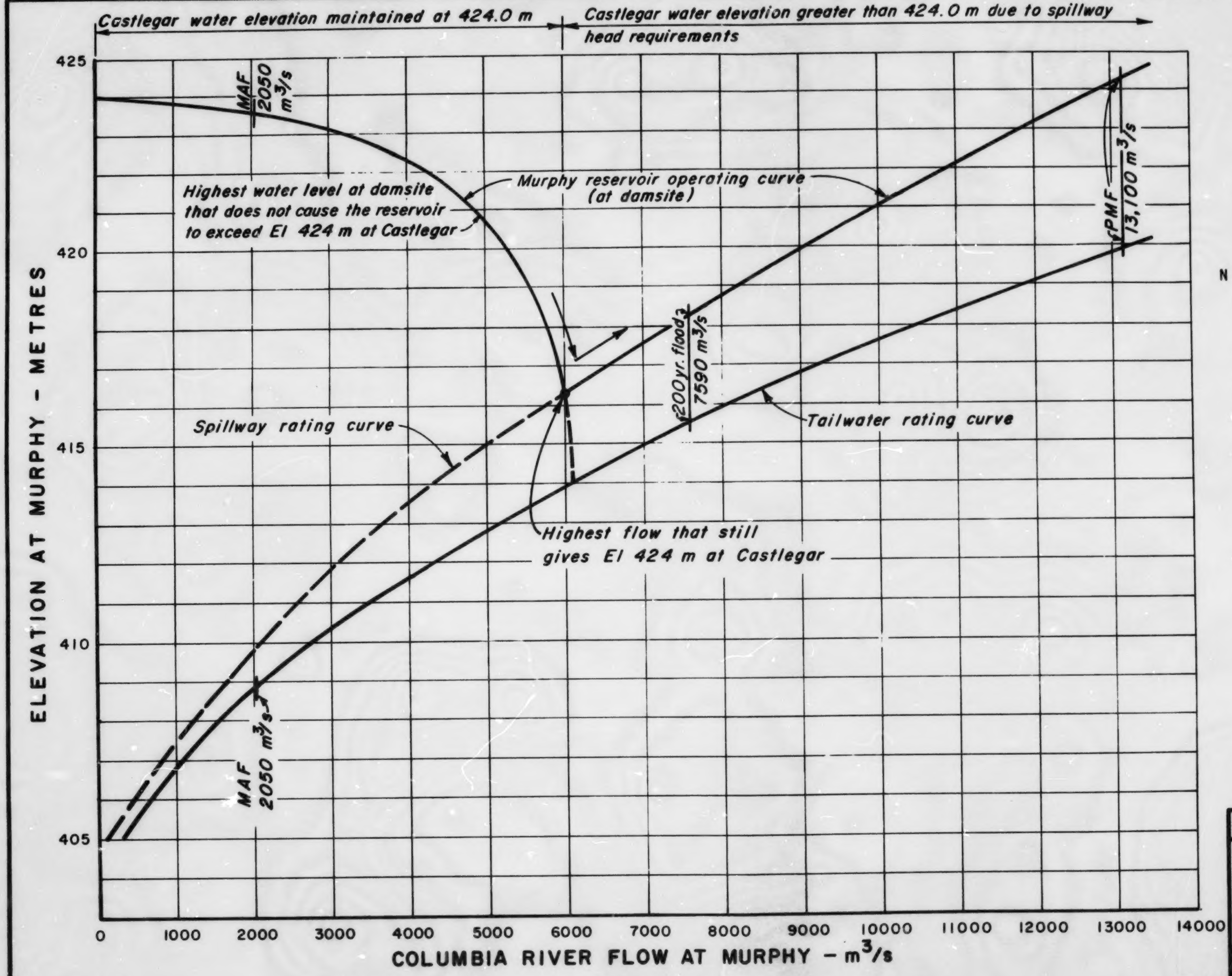
BACKWATER PROFILES

DATE MAY 1983

FIG. 5 - 2

DWN £

DWG. No. 218-C14-C604



NOTES:

1. FIVE 16.0 m SPILLWAY BAYS, CREST ELEVATION 404 m.
2. WATER LEVELS AT DAMSITE FOR FLOWS UP TO 6000 m^3/s BASED ON FLOW SPLIT 50% COLUMBIA AND 50% KOOTENAY RIVERS.
3. SPILLWAY RATING CURVE SHOWN IS TOTAL ENERGY GRADE MINUS THE VELOCITY HEAD FOR THE NATURAL RIVER SECTION.
4. CASTLEGAR WATER ELEVATIONS TAKEN AT WATER SURVEYS OF CANADA GAUGE #08NE002 AT CASTLEGAR FERRY.
5. FOR WATER LEVELS UPSTREAM OF DAM FOR 200 YEAR FLOOD AND PMF, SEE FIG. 5.2.

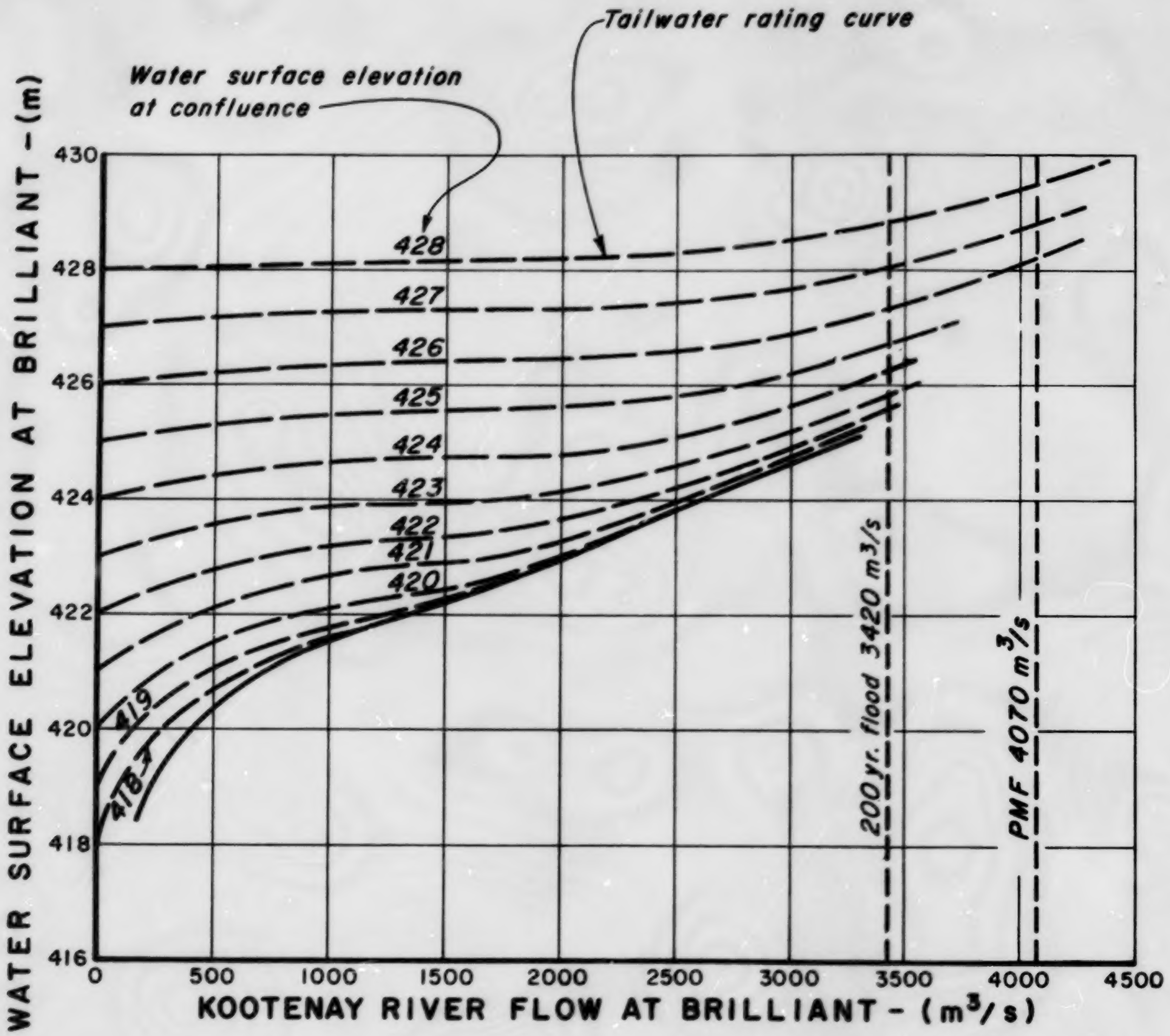
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

OPERATING CURVES

DATE MAY 1983	FIG. 5-3
DWN MWB	DWG. No. 218-C14-B605

REPORT No. H1617



NOTES:

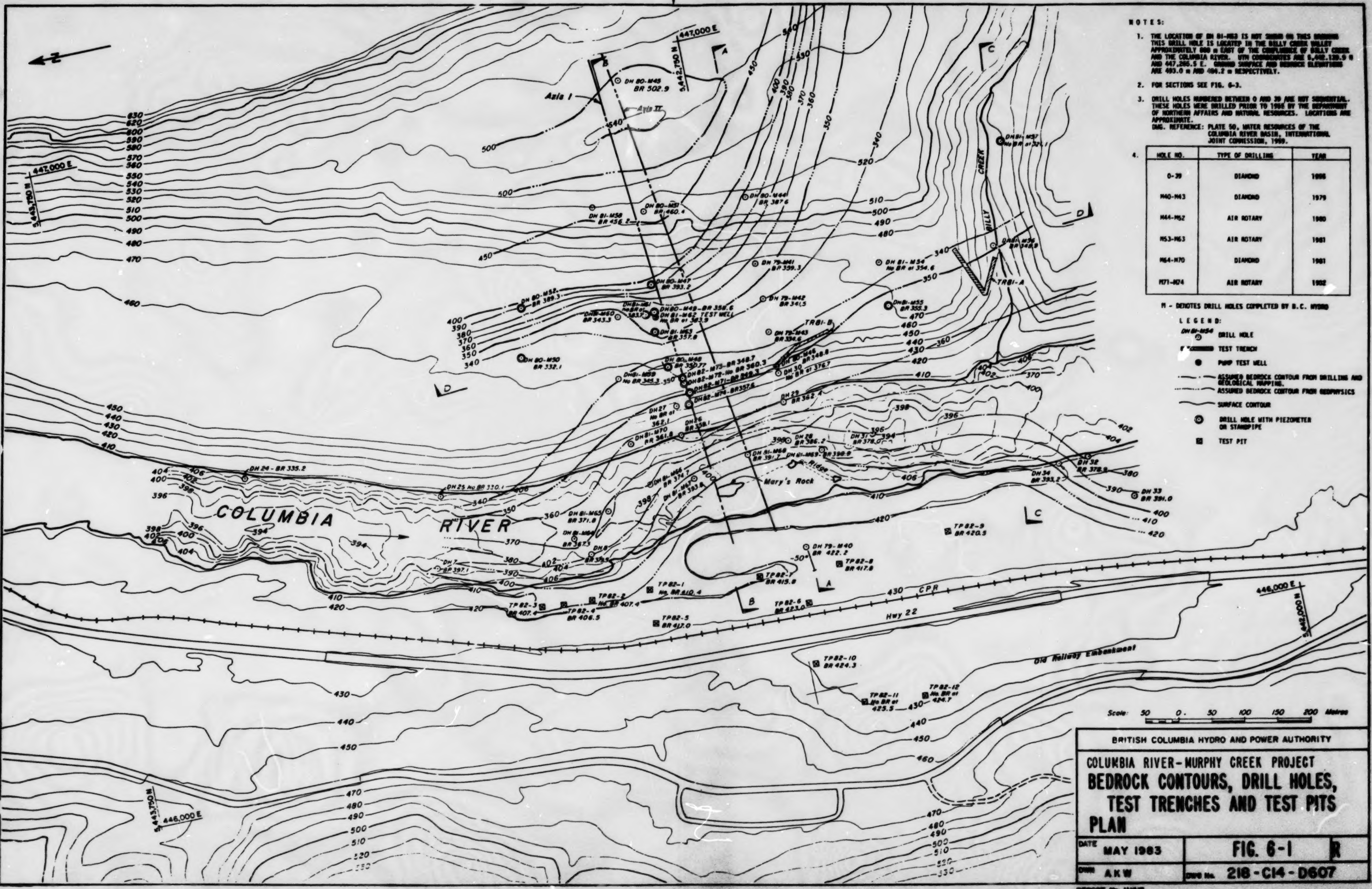
TAILWATER RATING CURVES AT BRILLIANT ARE SHOWN FOR VARYING WATER SURFACE ELEVATIONS AT THE CONFLUENCE OF THE COLUMBIA AND KOOTENAY RIVERS.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

TAILWATER RATING CURVES AT BRILLIANT

DATE MAY 1983	FIG. 5-4
DWN MWB	DWG. No. 218 - C14 - B606


N
447,000 E
5,442,750 N
Axis I
Axis II
DH 80-M45 BR 502.9
DH 80-M44 BR 387.6
DH 80-M51 BR 460.4
DH 81-M58 BR 456.2
DH 79-M41 BR 399.3
DH 81-M54 No BR at 354.6
DH 81-M57 No BR at 321.1
DH 81-M56 BR 348.9
DH 80-M47 BR 393.2
DH 80-M52 BR 389.3
DH 81-M60 BR 343.3
DH 80-M49 BR 358.6
DH 81-M62 TEST WELL
DH 81-M63 BR 357.8
DH 79-M42 BR 341.5
DH 81-M55 BR 355.3
DH 79-M43 BR 334.6
DH 80-M50 BR 332.1
DH 82-M73 No BR 348.7
DH 82-M72 BR 360.3
DH 82-M71 BR 357.6
DH 82-M70 BR 357.6
DH 81-M59 No BR 345.3
DH 80-M46 BR 348.8
DH 30 No BR at 376.7
DH 29 BR 362
DH 27 No BR at 362.1
DH 26 BR 358.1
DH 81-M70 BR 361.8
DH 81-M68 BR 391.7
DH 81-M69 BR 398.9
DH 28 BR 386.2
DH 31 BR 378.0
DH 32 BR 378.9
DH 34 BR 393.2
DH 33 BR 391.0
DH 24 - BR 335.2
DH 25 No BR 330.1
DH 81-M66 BR 376.7
DH 81-M67 BR 383.8
DH 81-M65 BR 371.8
DH 81-M64 BR 367.3
DH 8 BR 393
DH 7 BR 397.1
DH 79-M40 BR 422.2
TRBI-A
TRBI-B
BILLY CREEK
Mary's Rock
Ridge
COLUMBIA RIVER
TP 82-1 No BR 410.4
TP 82-2 No BR 407.4
TP 82-3 BR 407.4
TP 82-4 BR 406.5
TP 82-5 BR 417.0
TP 82-6 BR 423.0
TP 82-7 BR 415.8
TP 82-8 BR 417.8
TP 82-9 BR 420.5
TP 82-10 BR 424.3
TP 82-11 No BR at 425.5
TP 82-12 No BR at 424.7
CPR
Hwy 22
Old Railway Embankment
446,000 E
5,442,000 N
5,443,750 N
A
B
C
D
Scale: 50 0 50 100 150 200 Metres

NOTES:

1. THE LOCATION OF DH 81-M53 IS NOT SHOWN ON THIS DRAWING. THIS DRILL HOLE IS LOCATED IN THE BILLY CREEK VALLEY APPROXIMATELY 800 m EAST OF THE CONFLUENCE OF BILLY CREEK AND THE COLUMBIA RIVER. UTM COORDINATES ARE 5,442,120.9 N AND 447,285.5 E. GROUND SURFACE AND BEDROCK ELEVATIONS ARE 493.0 m AND 484.2 m RESPECTIVELY.

2. FOR SECTIONS SEE FIG. 6-3.

3. DRILL HOLES NUMBERED BETWEEN 0 AND 39 ARE NOT SEQUENTIAL. THESE HOLES WERE DRILLED PRIOR TO 1966 BY THE DEPARTMENT OF NORTHERN AFFAIRS AND NATURAL RESOURCES. LOCATIONS ARE APPROXIMATE.
DWG. REFERENCE: PLATE 50, WATER RESOURCES OF THE COLUMBIA RIVER BASIN, INTERNATIONAL JOINT COMMISSION, 1959.

4.

HOLE NO.	TYPE OF DRILLING	YEAR
0-39	DIAMOND	1966
M40-M43	DIAMOND	1979
M44-M52	AIR ROTARY	1980
M53-M63	AIR ROTARY	1981
M64-M70	DIAMOND	1981
M71-M74	AIR ROTARY	1982

M - DENOTES DRILL HOLES COMPLETED BY B.C. HYDRO

LEGEND:

DH 81-M54 ⊙	DRILL HOLE
▬	TEST TRENCH
●	PUMP TEST WELL
—·—	ASSUMED BEDROCK CONTOUR FROM DRILLING AND GEOLOGICAL MAPPING.
···—···	ASSUMED BEDROCK CONTOUR FROM GEOPHYSICS
~	SURFACE CONTOUR
◉	DRILL HOLE WITH PIEZOMETER OR STANDPIPE
▣	TEST PIT

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER-MURPHY CREEK PROJECT

BEDROCK CONTOURS, DRILL HOLES, TEST TRENCHES AND TEST PITS PLAN

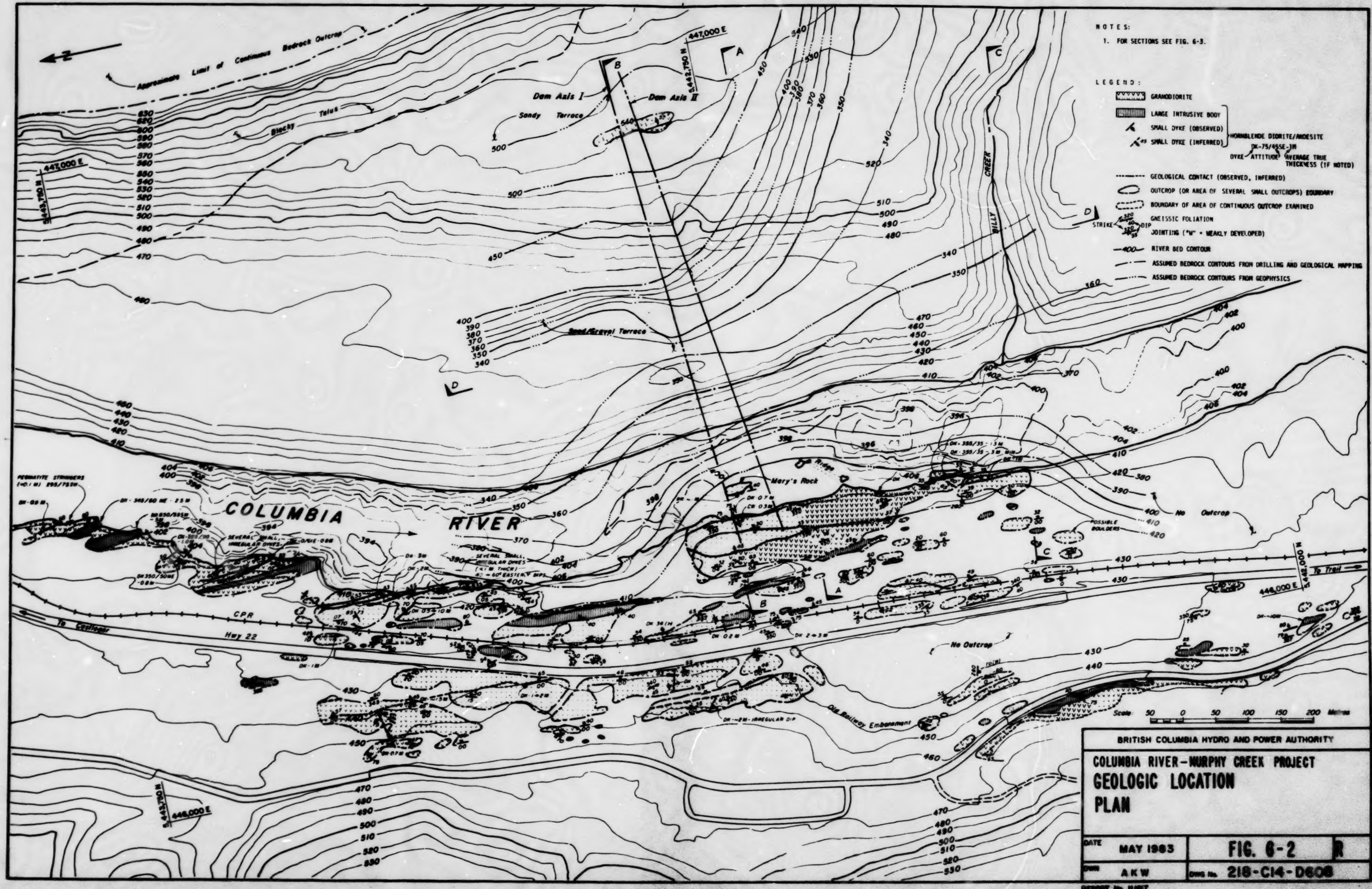
DATE MAY 1983

FIG. 6-1 R

DWN AKW

DWG No. 218-C14-D607

REPORT No. H1647


N
Approximate Limit of Continuous Bedrock Outcrop
Blocky Talus
Sandy Terrace
Dam Axis I
Dam Axis II
Sand/Gravel Terrace
447,000 E
5,442,750 N
446,000 E
5,442,000 N
A
B
C
D
BILLY CREEK
COLUMBIA RIVER
Mary's Rock
Ridge
No Outcrop
POSSIBLE BOULDERS
PEGMATITE STRINGERS (<0.1 M) 295/75SW
SEVERAL SMALL IRREGULAR DYKES
SEVERAL SMALL IRREGULAR DYKES (<1 M THICK) 47-60° EASTERLY DIPS
CPR
Hwy 22
To Castlegar
To Trail
Old Railway Embankment

NOTES:

1. FOR SECTIONS SEE FIG. 6-3.

LEGEND:

- ▨ GRANODIORITE
- ▥ LARGE INTRUSIVE BODY
- SMALL DYKE (OBSERVED)
- SMALL DYKE (INFERRED)

HORNBLENDE DIORITE/ANDESITE (large intrusive body, small dyke observed, small dyke inferred)

DK-75/45SE-1M
DYKE — ATTITUDE — AVERAGE TRUE THICKNESS (IF NOTED)

- GEOLOGICAL CONTACT (OBSERVED, INFERRED)
- OUTCROP (OR AREA OF SEVERAL SMALL OUTCROPS) BOUNDARY
- BOUNDARY OF AREA OF CONTINUOUS OUTCROP EXAMINED
- GNEISSIC FOLIATION
- JOINTING ("W" = WEAKLY DEVELOPED)

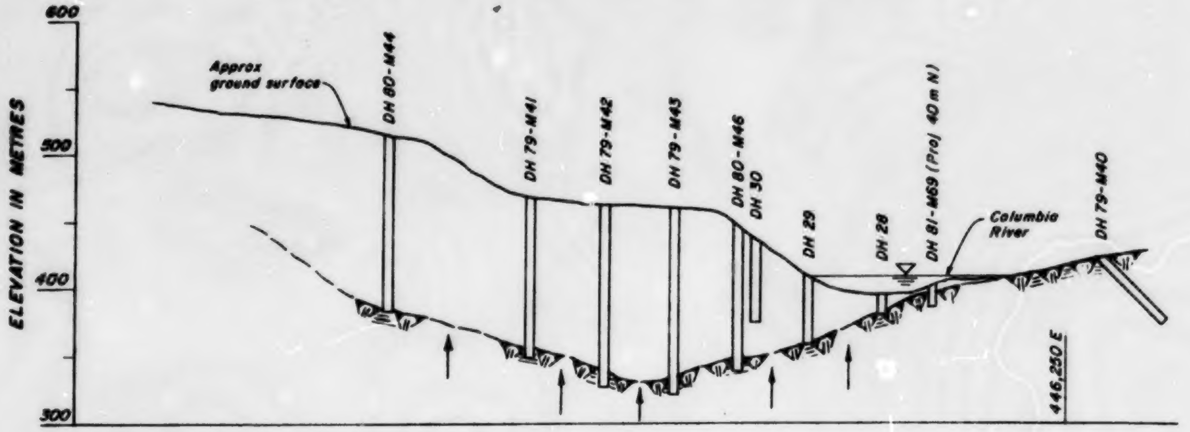
STRIKE — DIP

- —400— RIVER BED CONTOUR
- ASSUMED BEDROCK CONTOURS FROM DRILLING AND GEOLOGICAL MAPPING
- ASSUMED BEDROCK CONTOURS FROM GEOPHYSICS

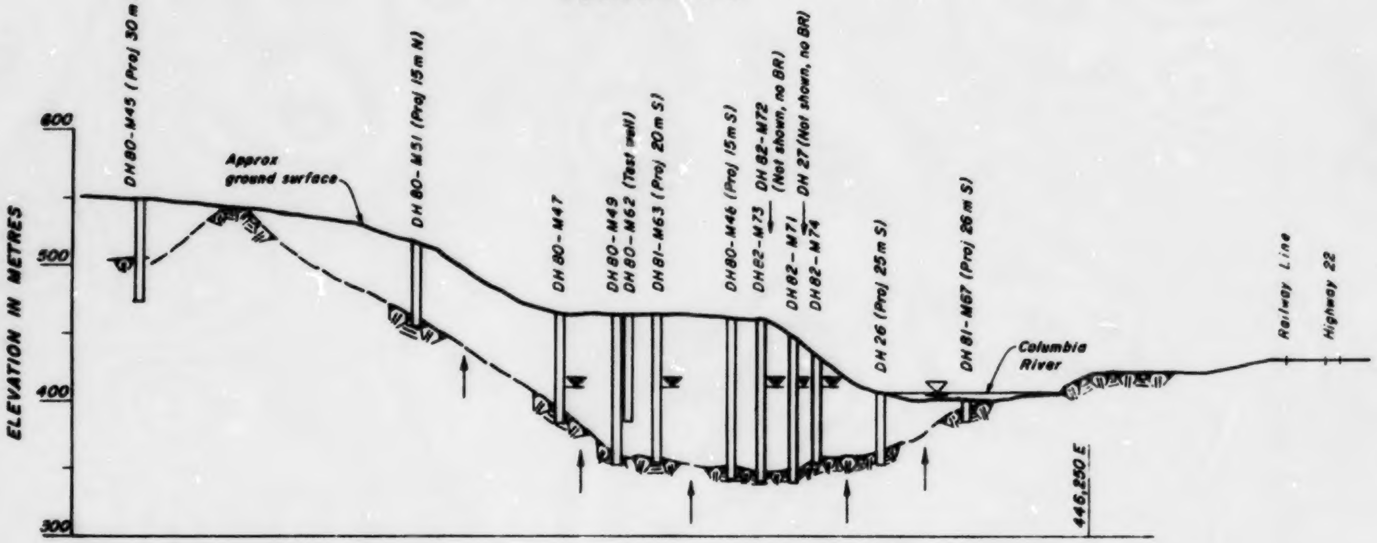
Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
COLUMBIA RIVER–MURPHY CREEK PROJECT GEOLOGIC LOCATION PLAN	
DATE MAY 1983	FIG. 6-2 R
DWN A K W	DWG No. 218-C14-D606

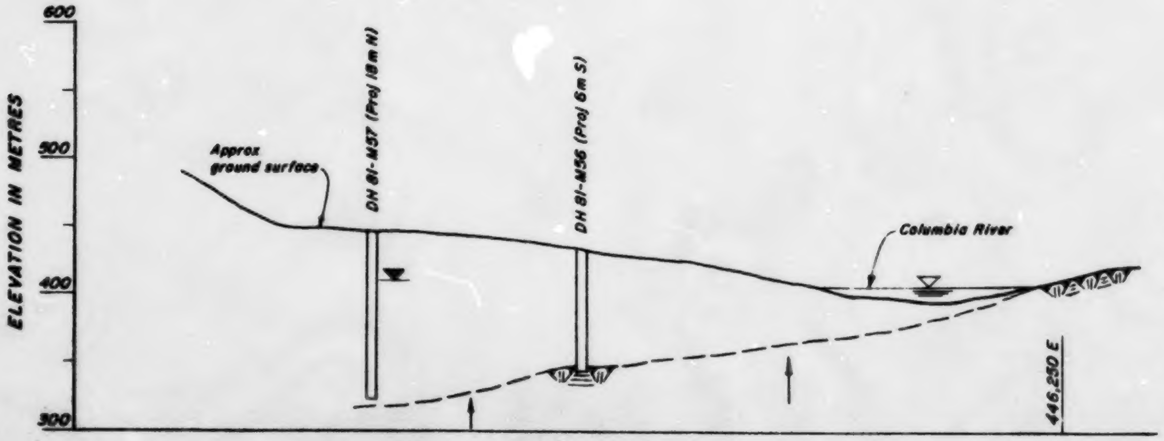
REPORT No. H1617


ELEVATION IN METRES
600
500
400
300
Approx ground surface
DH 80-M44
DH 79-M41
DH 79-M42
DH 79-M43
DH 80-M46
DH 30
DH 29
DH 28
DH 81-M69 (Proj 40 m N)
Columbia River
DH 79-M40
446,250 E

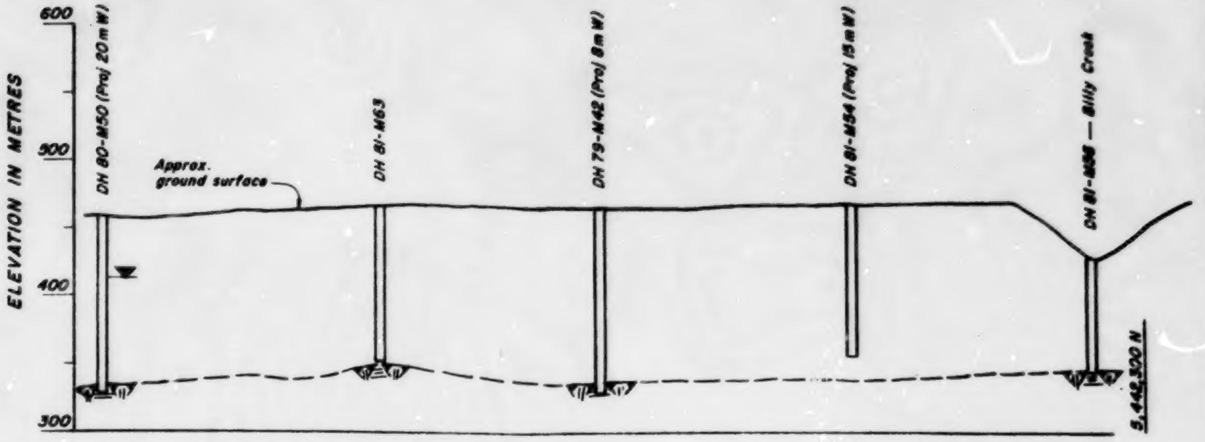
SECTION A-A


ELEVATION IN METRES
600
500
400
300
DH 80-M45 (Proj 30 m N)
Approx ground surface
DH 80-M51 (Proj 15 m N)
DH 80-M47
DH 80-M49
DH 80-M62 (Test well)
DH 81-M63 (Proj 20 m S)
DH 80-M46 (Proj 15 m S)
DH 82-M73
DH 82-M72 (Not shown, no BR)
DH 82-M71
DH 27 (Not shown, no BR)
DH 82-M74
DH 26 (Proj 25 m S)
DH 81-M67 (Proj 26 m S)
Columbia River
Railway Line
Highway 22
446,250 E

SECTION B-B


ELEVATION IN METRES
600
500
400
300
Approx ground surface
DH 81-M57 (Proj 18 m N)
DH 81-M56 (Proj 6 m S)
Columbia River
446,250 E

SECTION C-C


ELEVATION IN METRES
600
500
400
300
DH 80-M50 (Proj 20 m W)
Approx. ground surface
DH 81-M63
DH 79-M42 (Proj 8 m W)
DH 81-M54 (Proj 15 m W)
DH 81-M56 — Billy Creek
5,442,300 N

SECTION D-D

NOTES:

1. OVERBURDEN CONSISTS PREDOMINANTLY OF SAND AND GRAVEL WITH FREQUENT BOULDERS.
2. BEDROCK CONSISTS PREDOMINANTLY OF GRANODIORITE.
3. FOR LOCATION OF SECTIONS SEE FIG. 6-1 AND 6-2.

LEGEND:

APPROXIMATE BEDROCK FROM DRILLING DATA & FIELD MAPPING

▼ PIEZOMETER LEVEL, AUG. 4, 1982.

— — — APPROXIMATE BEDROCK FROM GEOPHYSICAL DATA.

↑ LOCATION OF POSSIBLE CHANNELS

Scale: 50 0 50 100 150 200 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER-MURPHY CREEK PROJECT

GEOLOGIC SECTIONS

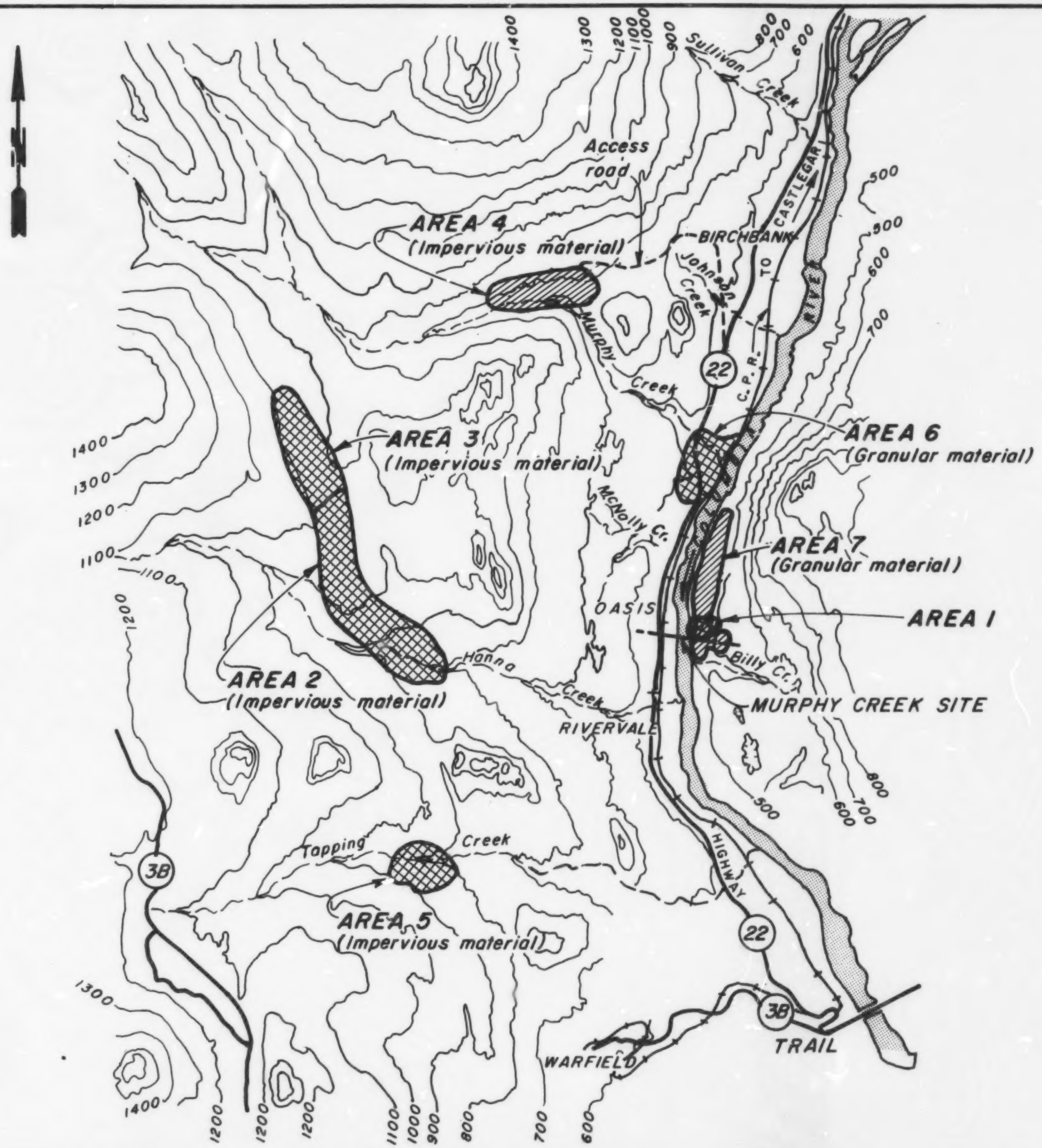
DATE MAY 1983

FIG. 6-3

DWN AKW

DWG No. 218-C14-D609

REPORT No. H1617


N
1400
1300
1200
1100
1000
900
Sullivan Creek
800
700
600
Access road
CASTLEGAR
TO
BIRCHBANK
AREA 4
(Impervious material)
Johnson Creek
Murphy Creek
RIVER
22
C.P.R.
500
500
600
700
AREA 3
(Impervious material)
AREA 6
(Granular material)
McNally Cr.
1400
1300
1200
1100
1100
1200
AREA 7
(Granular material)
OASIS
AREA I
Billy Cr.
AREA 2
(Impervious material)
Hanna Creek
MURPHY CREEK SITE
RIVERVALE
500
600
700
800
HIGHWAY
Tapping Creek
3B
AREA 5
(Impervious material)
22
1300
3B
TRAIL
WARFIELD
1400
1200
1200
1200
1100
1000
900
800
700
600

NOTES:

1. AREA 1 IS CONTAINED WITHIN AREA 7.
2. THE OTHER AREAS EXPLORED FORMED PART OF THE OVERALL CONSTRUCTION MATERIALS EXPLORATION PROGRAM, RESULTING IN THE POTENTIAL BORROW AREAS IDENTIFIED.

LEGEND:

▨	POTENTIAL BORROW AREAS
▦	OTHER AREAS EXPLORED

Scale 1000 0 1000 2000 Metre

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

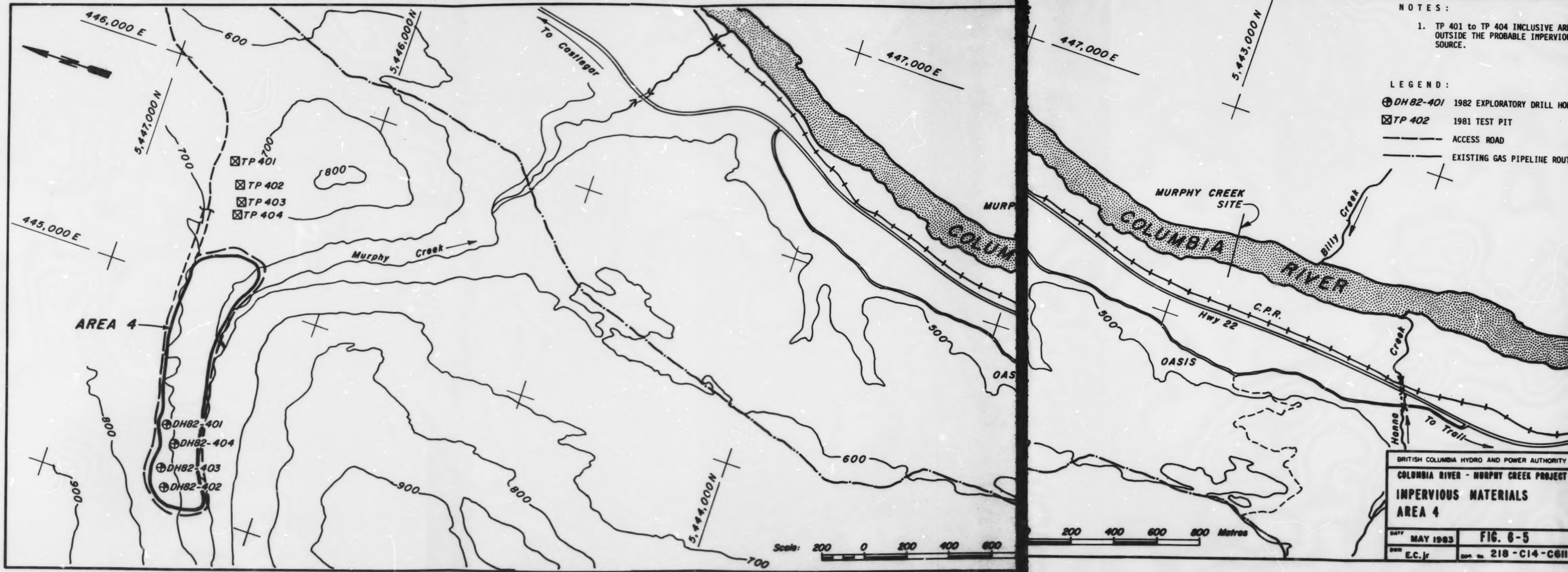
CONSTRUCTION MATERIALS AREA LOCATION PLAN

DATE MAY 1983

FIG. 6-4

DWN E.C.gr.

218-C14-B610


446,000 E
600
To Castlegar
5,446,000 N
5,447,000 N
700
700
TP 401
TP 402
TP 403
TP 404
800
445,000 E
Murphy Creek
AREA 4
DH82-401
DH82-404
DH82-403
DH82-402
800
900
900
800
5,444,000 N
600
700
500
OAS
MURP
COLUM
447,000 E
447,000 E
5,443,000 N
MURPHY CREEK SITE
COLUMBIA RIVER
Billy Creek
C.P.R.
Hwy 22
500
OASIS
Creek
Hanna
To Trail
Scale: 200 0 200 400 600 800 Metres

NOTES:

1. TP 401 to TP 404 INCLUSIVE AR
OUTSIDE THE PROBABLE IMPERVIO
SOURCE.

LEGEND:

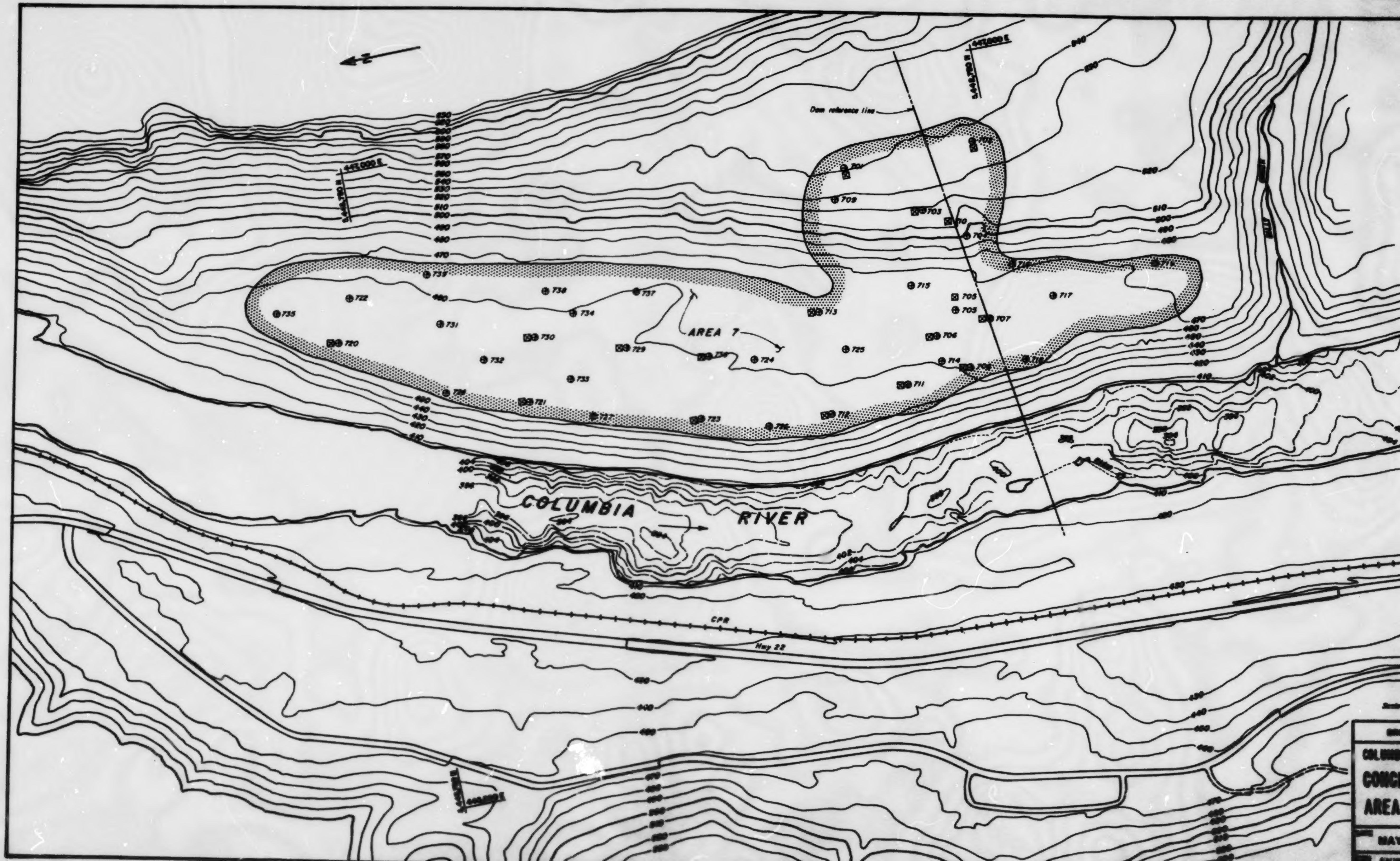
⊕ DH 82-401	1982 EXPLORATORY DRILL HO
☒ TP 402	1981 TEST PIT
— — — —	ACCESS ROAD
——·——	EXISTING GAS PIPELINE ROU

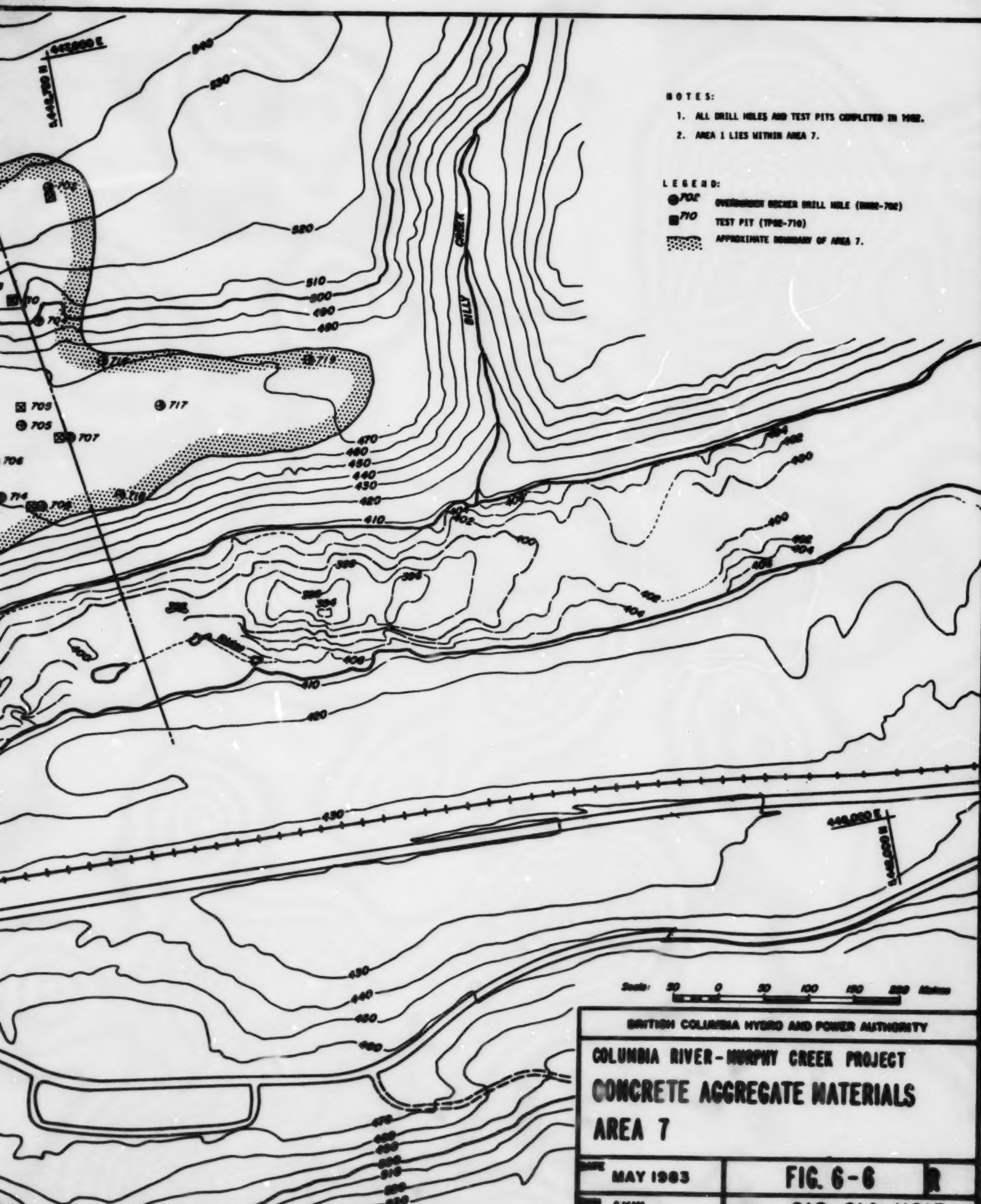
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

IMPERVIOUS MATERIALS
AREA 4

DATE MAY 1983
FIG. 6-5
DRN E.C.jr
DWG. No. 218-C14-C611


N
442,000 E
5,448,750 N
Dam reference line
AREA 7
COLUMBIA
RIVER
CPR
Hwy 22
701
702
703
704
705
706
707
708
709
710
711
712
713
714
715
716
717
718
719
720
721
722
723
724
725
726
727
728
729
730
731
732
733
734
735
736
737
738
739
840
830
820
810
800
490
480
470
460
450
440
430
420
410
400
396
COLUMB
CONC
AREA
MAY


540
530
520
510
500
490
480
470
460
450
440
430
420
410
400
406
404
402
398
396
394
392
430
420
410
440
450
460
470
480
490
500
510
520
530
BILLY CREEK
702
710
704
716
719
705
705
717
707
706
714
708
718
Scale: 50 0 50 100 150 200 Metres

NOTES:

1. ALL DRILL HOLES AND TEST PITS COMPLETED IN 1982.
2. AREA 1 LIES WITHIN AREA 7.

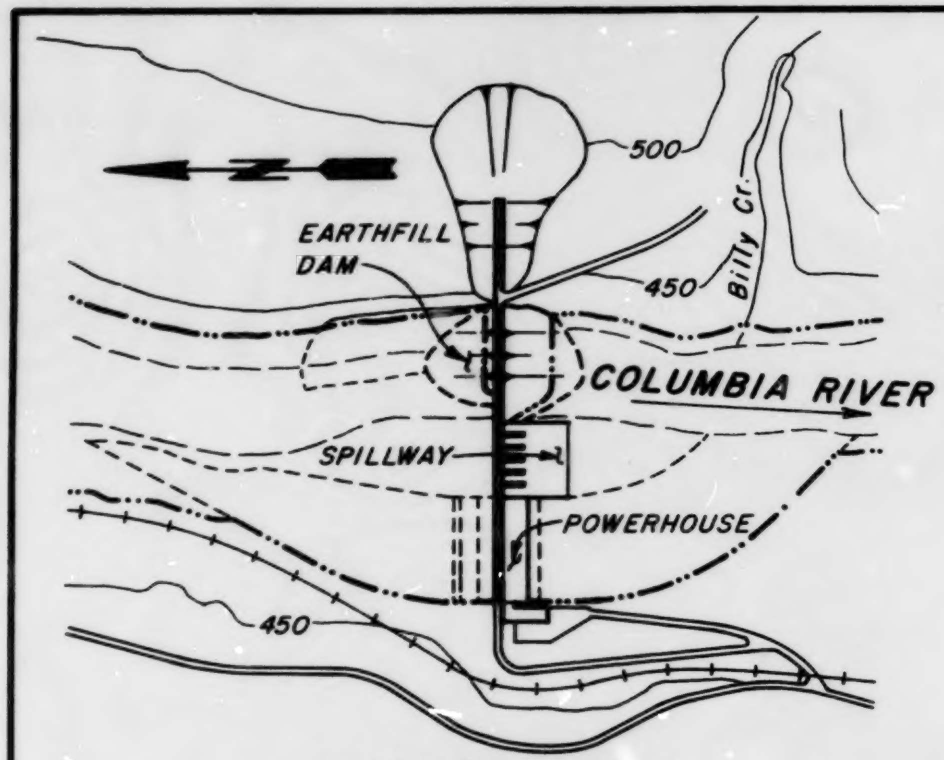
LEGEND:

⊕702 OVERBURDEN BECKER DRILL HOLE (BH82-702)
⊠710 TEST PIT (TP82-710)
▨ APPROXIMATE BOUNDARY OF AREA 7.

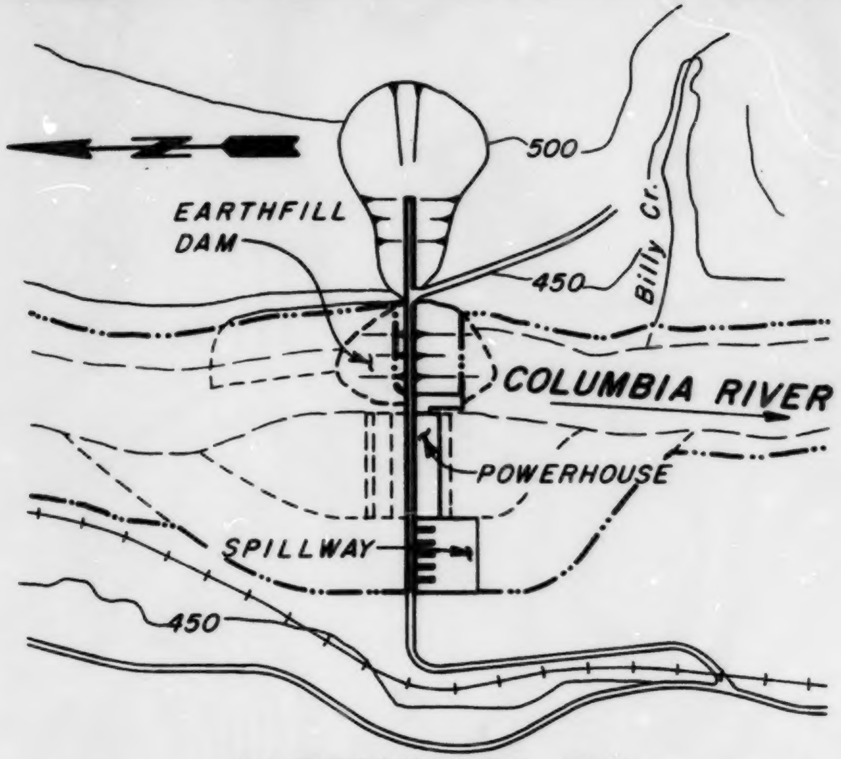
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER-MURPHY CREEK PROJECT
CONCRETE AGGREGATE MATERIALS
AREA 7

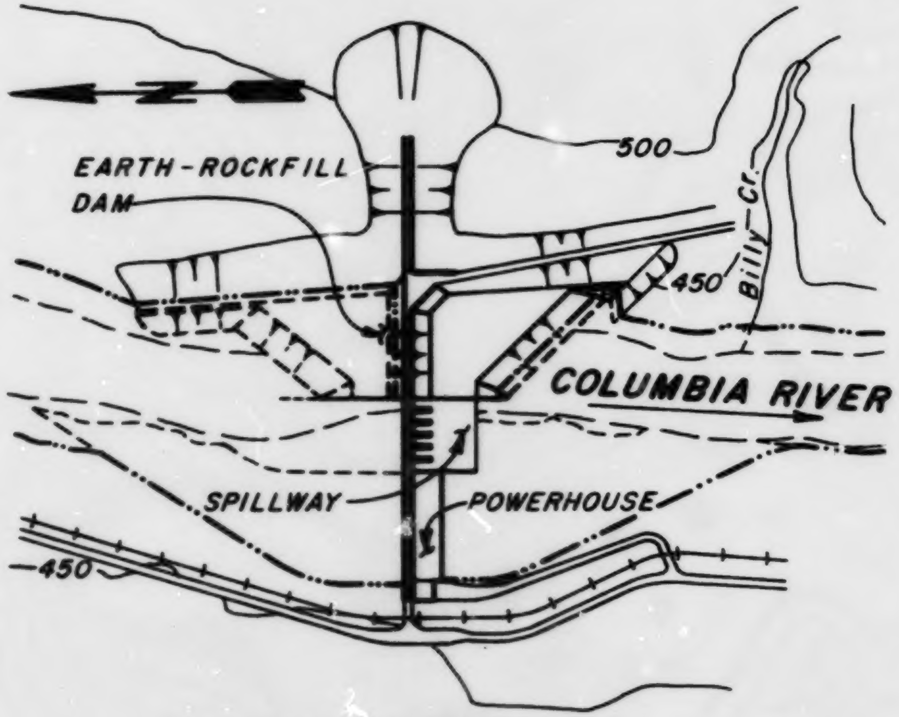
DATE MAY 1983	FIG. 6-6 R
AKW	DWG No. 218-C14-U613


N
500
450
450
Billy Cr.
EARTHFILL DAM
COLUMBIA RIVER
SPILLWAY
POWERHOUSE

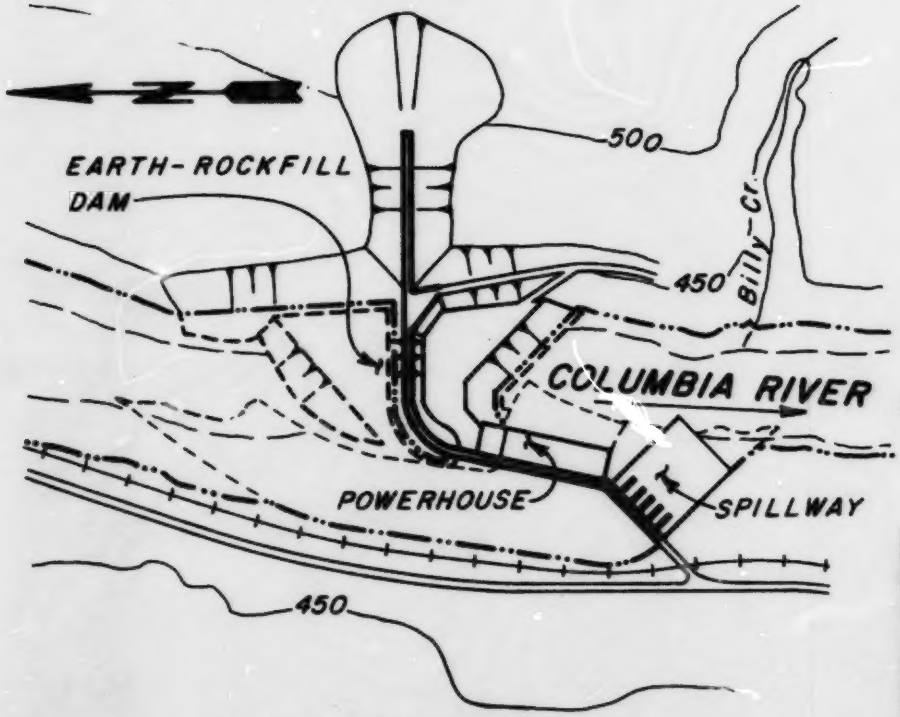
ALTERNATIVE F.1
EARTHFILL DAM


N
500
450
450
Billy Cr.
EARTHFILL DAM
COLUMBIA RIVER
POWERHOUSE
SPILLWAY

ALTERNATIVE F.2
EARTHFILL DAM


N
500
450
450
Billy Cr.
EARTH-ROCKFILL DAM
COLUMBIA RIVER
SPILLWAY
POWERHOUSE

ALTERNATIVE F.3
EARTH-ROCKFILL DAM
(Cross channel)


N
500
450
450
Billy Cr.
EARTH-ROCKFILL DAM
COLUMBIA RIVER
POWERHOUSE
SPILLWAY

ALTERNATIVE F.4
EARTH-ROCKFILL DAM
(Side channel)

Scale: 100 0 100 200
Metres

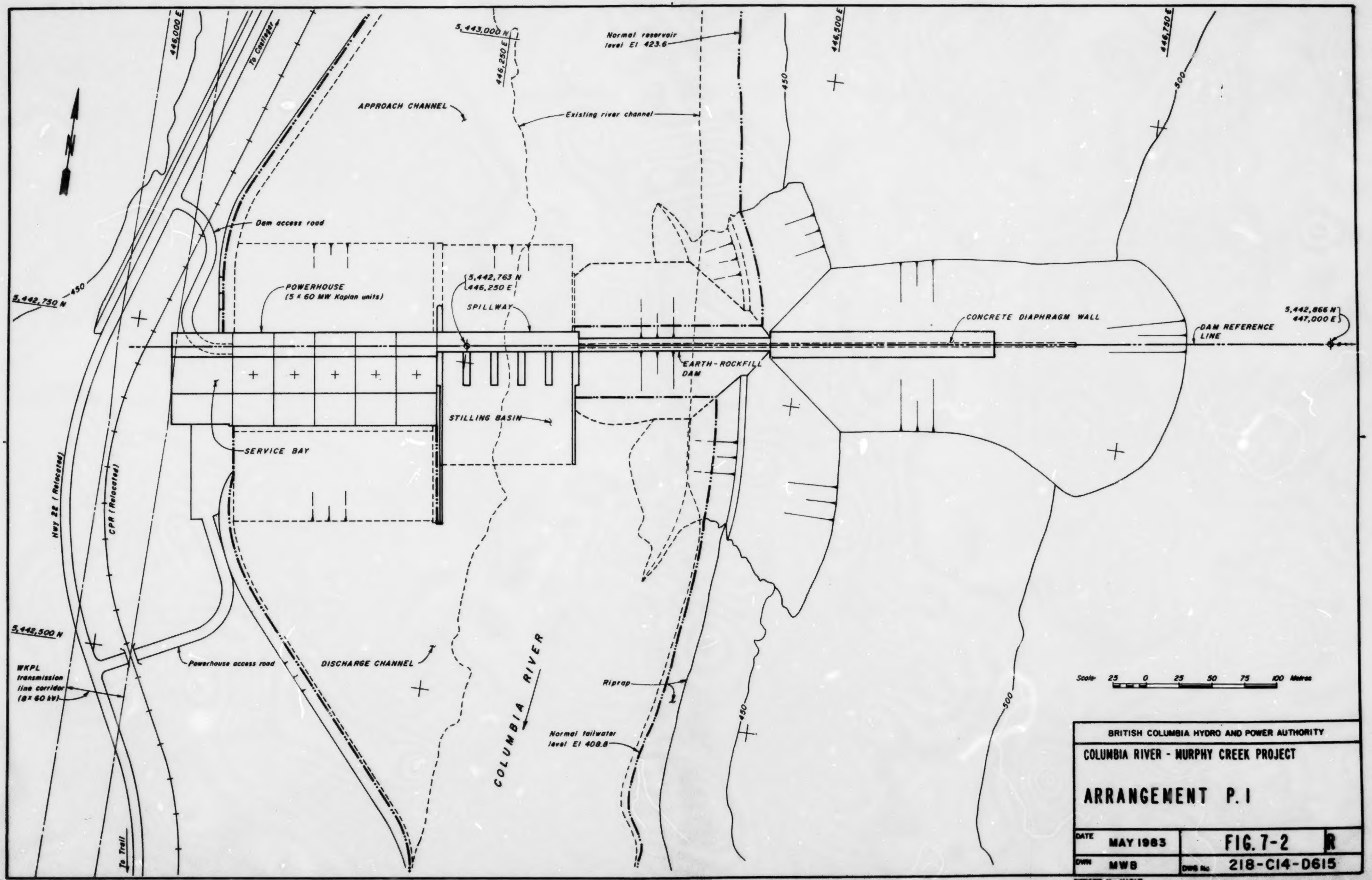
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

ALTERNATIVE ARRANGEMENTS

DATE MAY 1983	FIG. 7-1
DWN MWB	DWG. No. 218-C14-B614

REPORT No. H1617


446,000 E
To Castlegar
5,443,000 N
446,250 E
Normal reservoir level El 423.6
446,500 E
446,750 E
N
450
500
APPROACH CHANNEL
Existing river channel
Dam access road
5,442,750 N
450
POWERHOUSE (5 x 60 MW Kaplan units)
5,442,763 N 446,250 E
SPILLWAY
CONCRETE DIAPHRAGM WALL
5,442,866 N 447,000 E
DAM REFERENCE LINE
EARTH-ROCKFILL DAM
STILLING BASIN
SERVICE BAY
Hwy 22 (Relocated)
CPR (Relocated)
5,442,500 N
WKPL transmission line corridor (8 x 60 kV)
Powerhouse access road
DISCHARGE CHANNEL
COLUMBIA RIVER
Riprap
Normal tailwater level El 408.8
450
500
To Trail
Scale 25 0 25 50 75 100 Metres

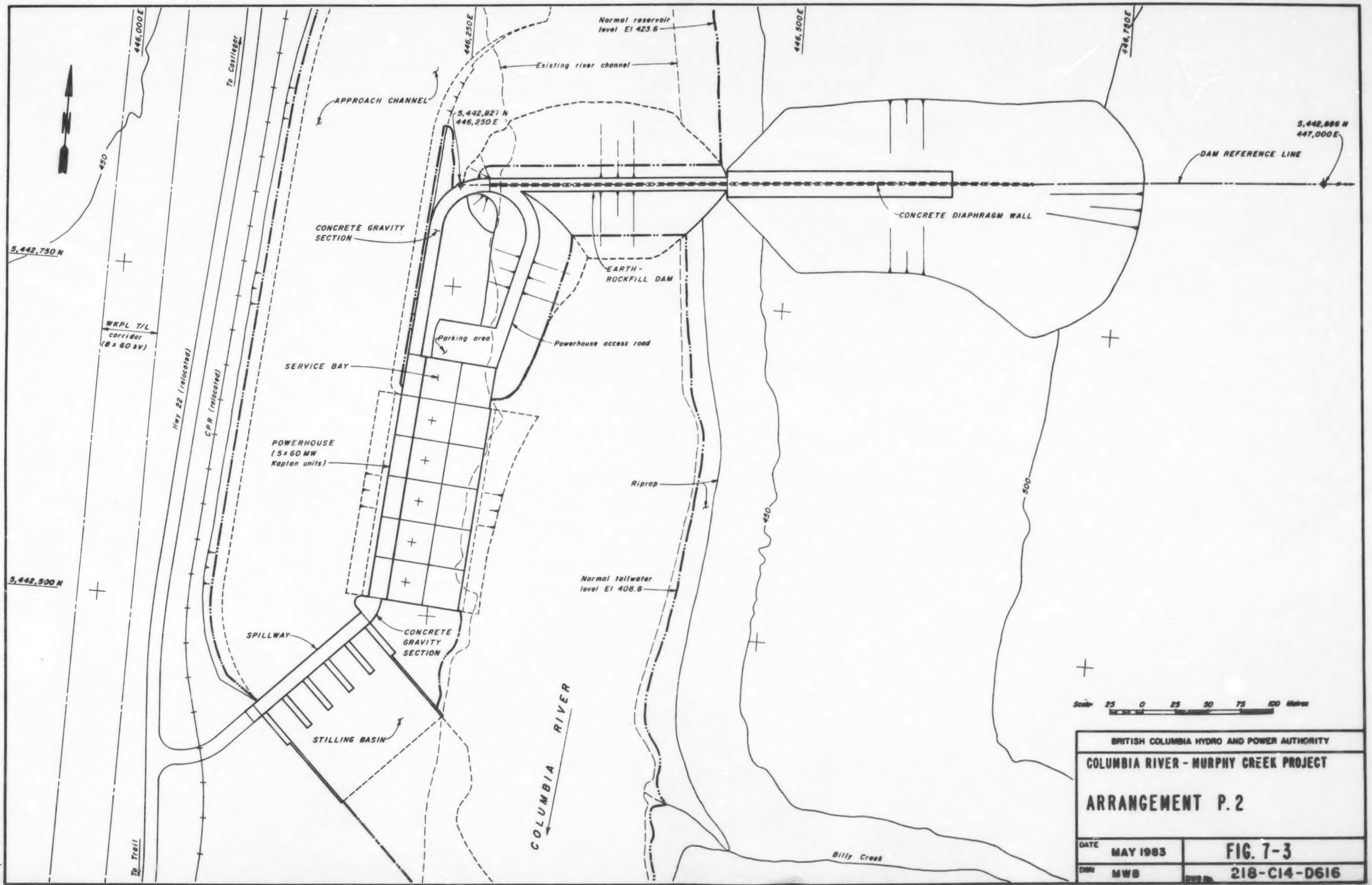
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

ARRANGEMENT P. 1

DATE MAY 1983
FIG. 7-2 R
DWN MWB
DWG No. 218-C14-D615

REPORT No. H1617


N
446,000 E
To Castlegar
446,250 E
Normal reservoir level El 423.6
Existing river channel
446,500 E
446,750 E
APPROACH CHANNEL
5,442,827 N
446,230 E
5,442,886 N
447,000 E
DAM REFERENCE LINE
450
CONCRETE DIAPHRAGM WALL
CONCRETE GRAVITY SECTION
5,442,750 N
EARTH-ROCKFILL DAM
WKPL T/L corridor (8 x 60 kV)
Parking area
Powerhouse access road
SERVICE BAY
Hwy 22 (relocated)
CPR (relocated)
POWERHOUSE (5 x 60 MW Kaplan units)
Riprap
500
450
5,442,500 N
Normal tailwater level El 408.8
SPILLWAY
CONCRETE GRAVITY SECTION
COLUMBIA RIVER
STILLING BASIN
To Trail
Billy Creek
Scale 25 0 25 50 75 100 Metres

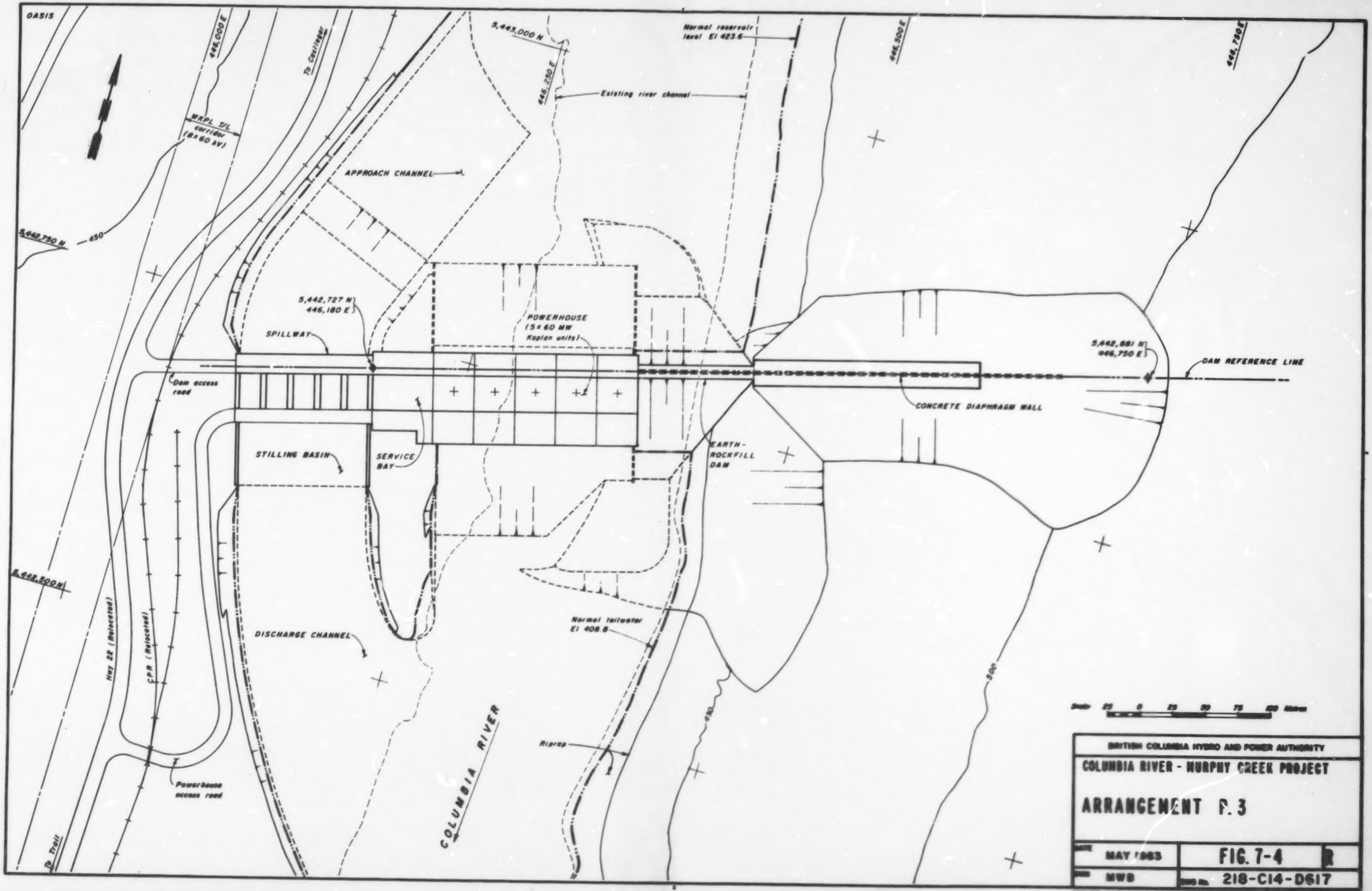
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

ARRANGEMENT P. 2

DATE MAY 1983	FIG. 7-3
DWN MWB	DWG No. 218-C14-D616

REPORT No. [illegible]


OASIS
446,000 E
To Castlegar
5,443,000 N
446,230 E
Existing river channel
Normal reservoir level El 423.6
446,300 E
446,750 E
WKPL T/L corridor (8×60 kV)
APPROACH CHANNEL
5,442,750 N
450
5,442,727 N
446,180 E
SPILLWAY
POWERHOUSE (5×60 MW Kaplan units)
5,442,661 N
446,750 E
DAM REFERENCE LINE
Dam access road
CONCRETE DIAPHRAGM WALL
STILLING BASIN
SERVICE BAY
EARTH-ROCKFILL DAM
5,442,500 N
Normal tailwater El 408.6
Hwy 22 (Relocated)
CPR (Relocated)
DISCHARGE CHANNEL
500
COLUMBIA RIVER
Riprap
430
Powerhouse access road
To Trail
Scale 25 0 25 50 75 100 Metres

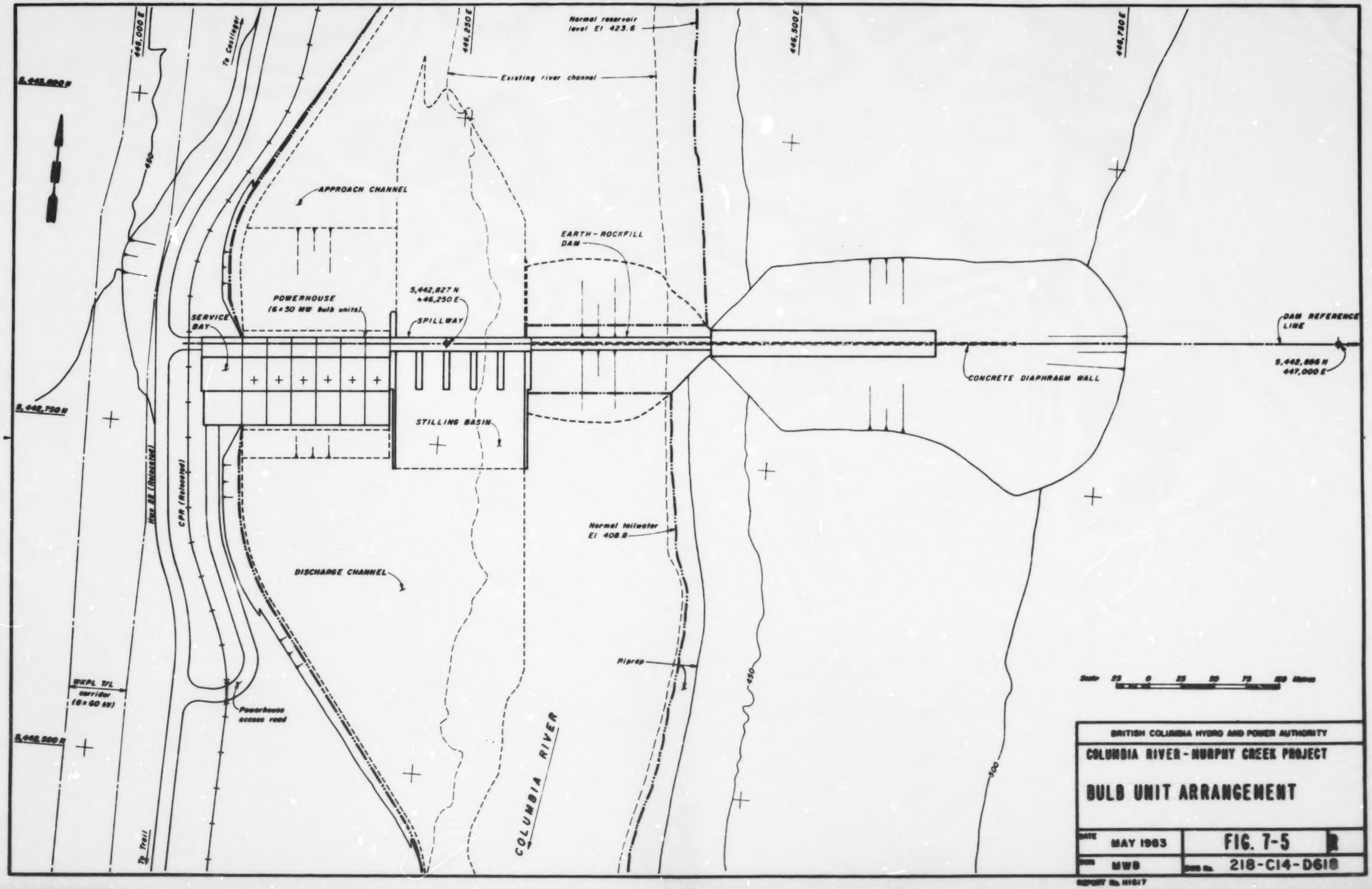
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

ARRANGEMENT P.3

DATE MAY 1983	FIG. 7-4 R
MWB	DWG No. 218-C14-D617

REPORT No. [illegible]


5,442,000 N
446,000 E
To Castlegar
446,250 E
Normal reservoir level El 423.6
446,500 E
446,750 E
Existing river channel
450
APPROACH CHANNEL
EARTH-ROCKFILL DAM
POWERHOUSE
(6×50 MW bulb units)
5,442,827 N
446,250 E
SERVICE BAY
SPILLWAY
DAM REFERENCE LINE
5,442,896 N
447,000 E
CONCRETE DIAPHRAGM WALL
5,442,750 N
STILLING BASIN
Hwy 22 (Relocated)
CPR (Relocated)
Normal tailwater El 408.8
DISCHARGE CHANNEL
Riprap
450
WKPL T/L corridor (6×60 kV)
Powerhouse access road
5,442,500 N
COLUMBIA RIVER
500
To Trail
Scale 25 0 25 50 75 100 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

BULB UNIT ARRANGEMENT

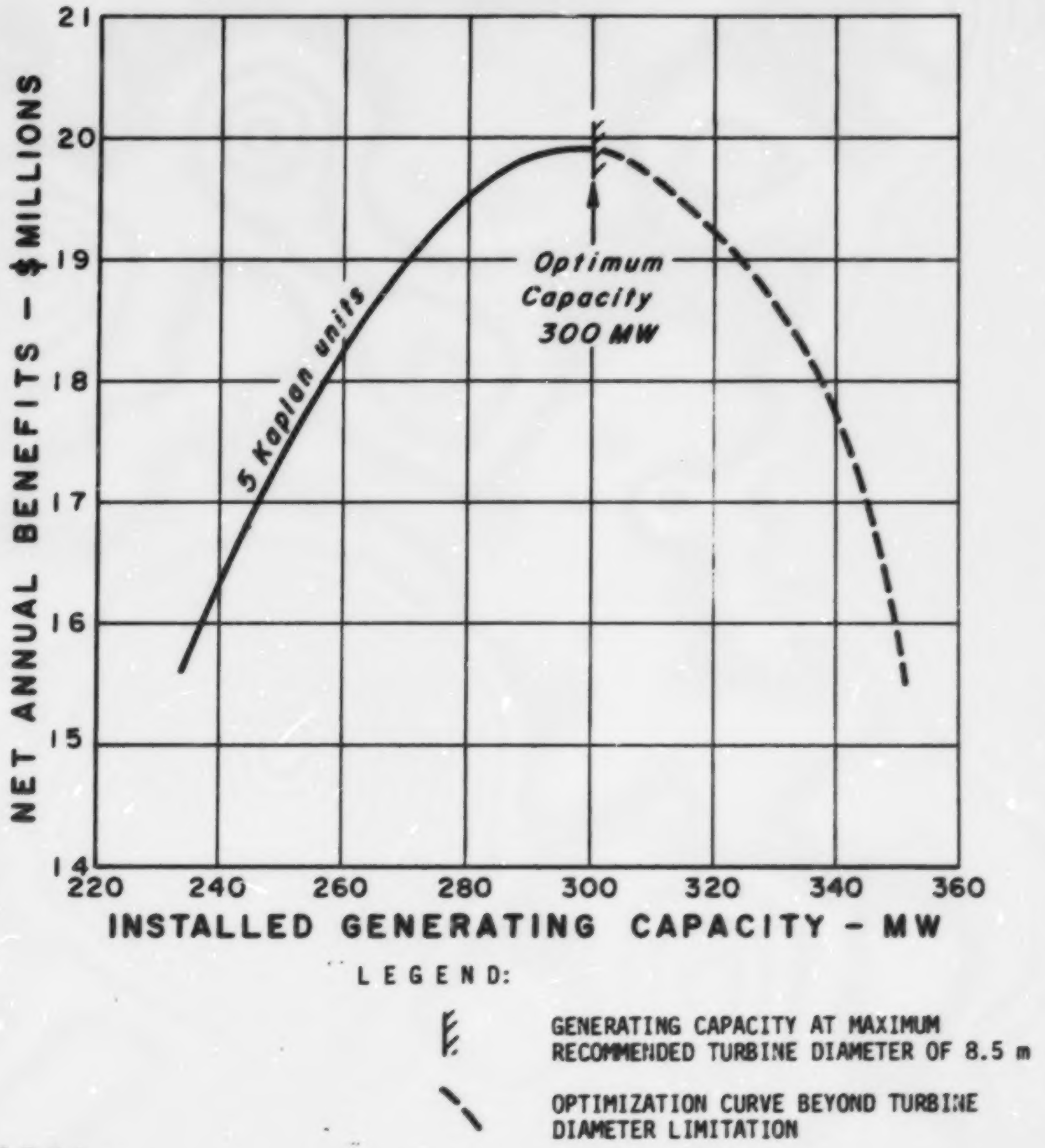
DATE MAY 1983

FIG. 7-5 R

MWB

DWG No. 218-C14-D618

REPORT No. H1617



NOTES:

1. NET ANNUAL BENEFITS BASED ON OCTOBER 1981 PRICE LEVELS.
2. ANNUAL WATER LICENCE FEES AND PROPERTY TAXES HAVE NOT BEEN INCLUDED IN THE ANNUAL COSTS.
3. NET ANNUAL BENEFITS ARE THE DIFFERENCE BETWEEN ANNUAL BENEFITS AND ANNUAL COSTS.
4. LOSSES AT BRILLIANT HAVE BEEN SUBTRACTED FROM ANNUAL BENEFITS.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

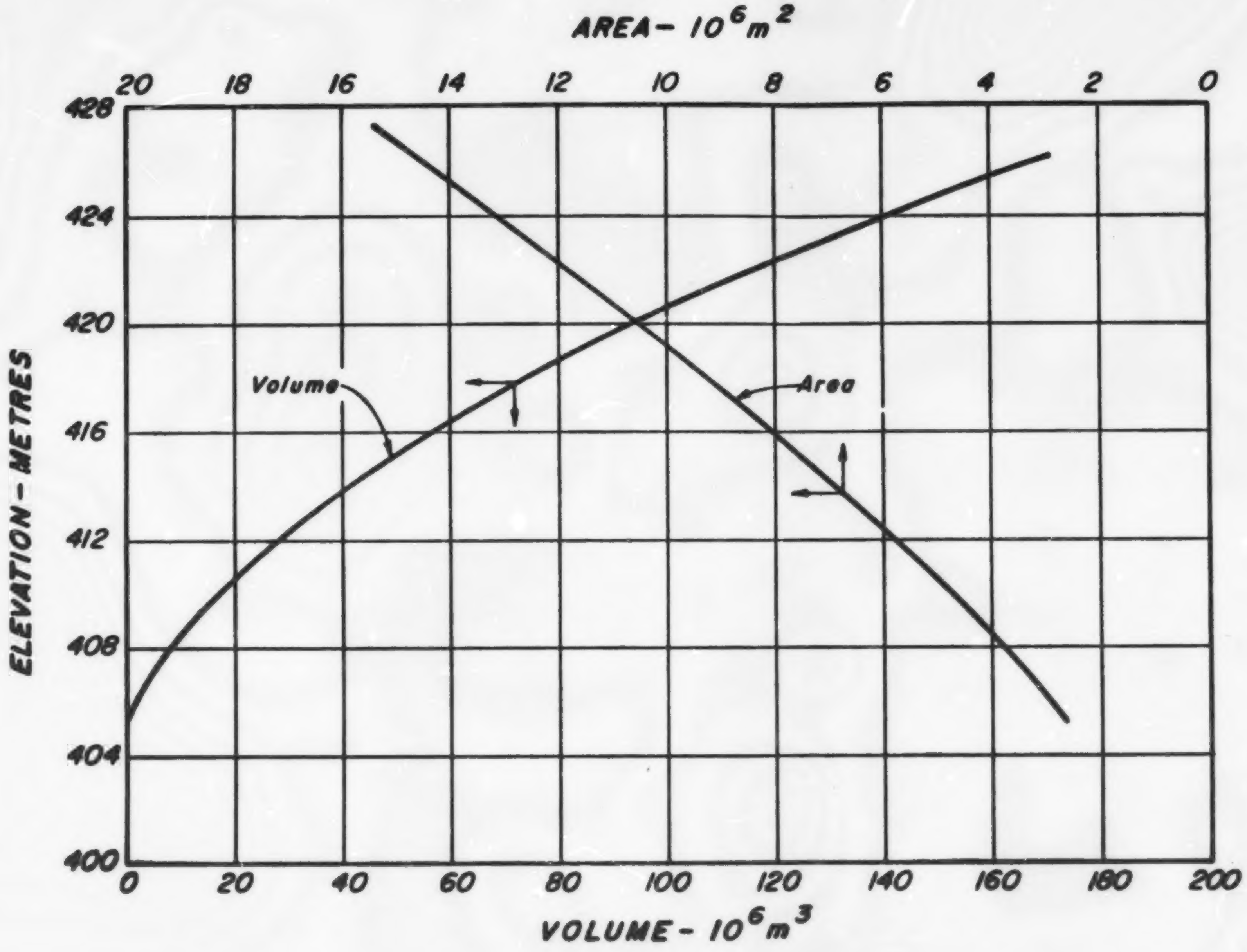
CAPACITY OPTIMIZATION

DATE MAY 1983	FIG. 7-6
DWN MWB	DWG. No 218-C14-A619

M

REPORT No H1617

ECTRIC DESIGN DIVISION A-R



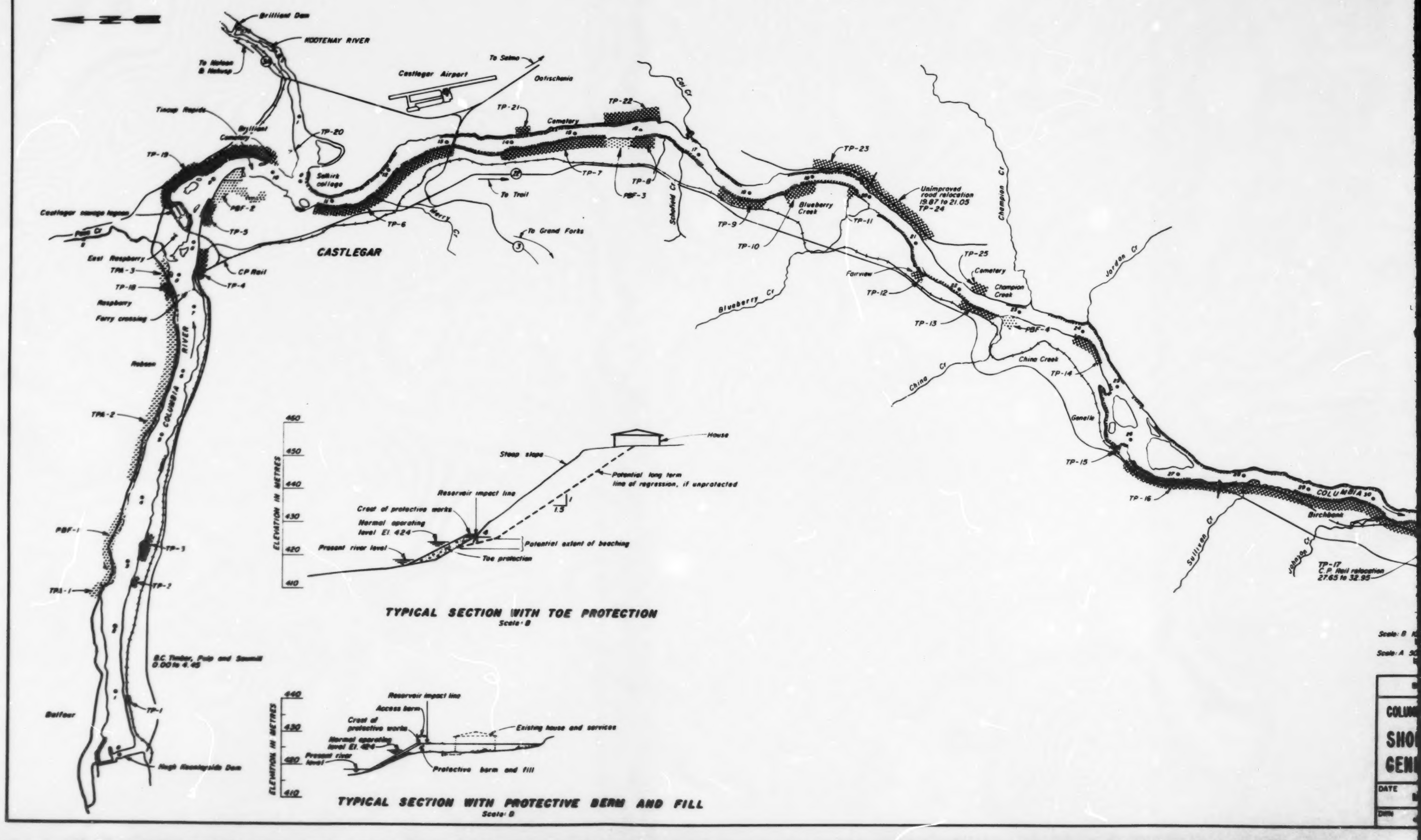
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

RESERVOIR AREA AND VOLUME
CURVES

DATE MAY 1983	FIG. 8-1
DWN M.O.	DWG. No 218-C14-B620

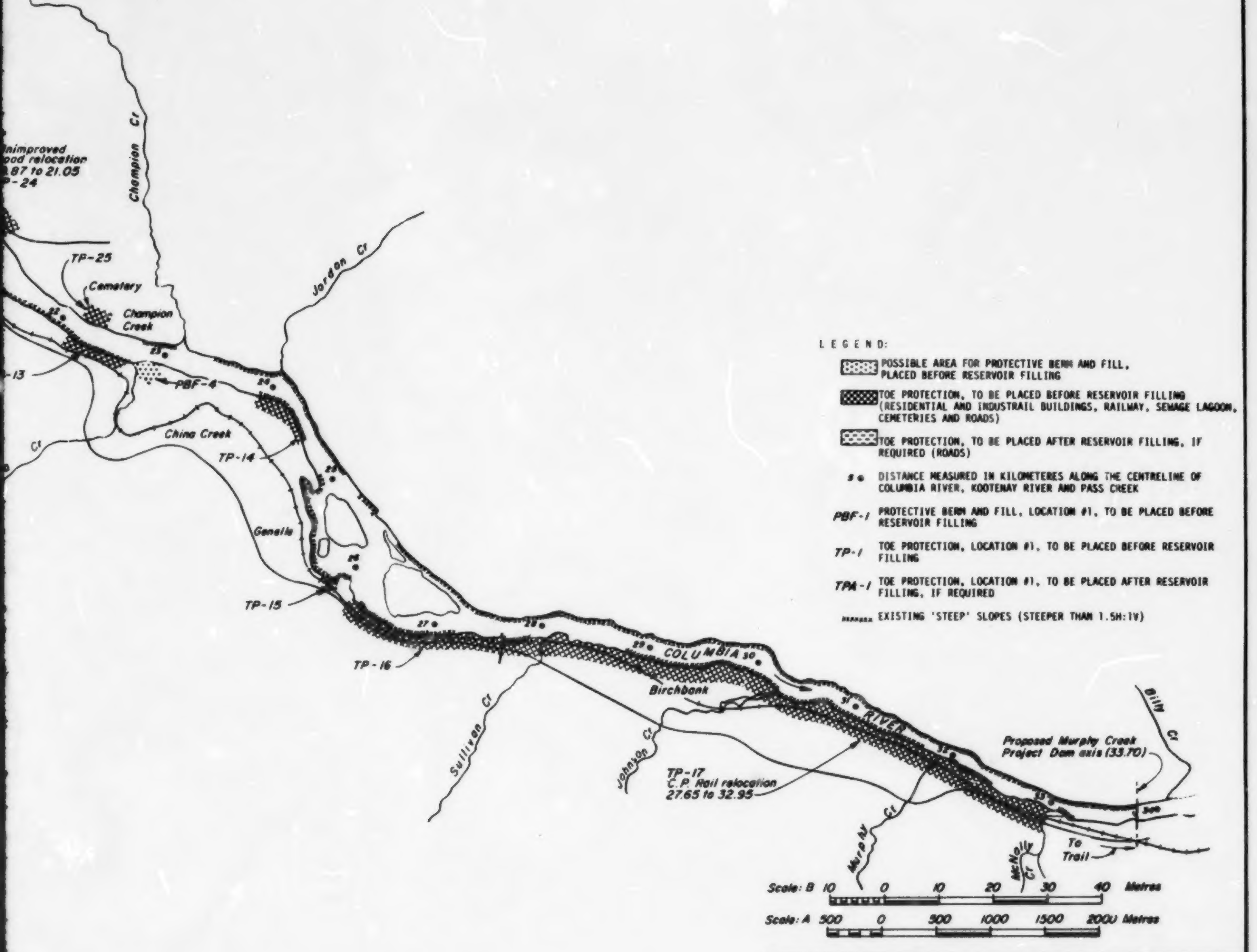
REPORT No H1617


N
Brilliant Dam
KOOTENAY RIVER
To Nelson & Nakusp
To Salmo
Castlegar Airport
Ootischenia
Tincup Rapids
Brilliant
Cemetery
TP-19
TP-20
Selkirk college
TP-21
Cemetery
TP-22
Cai Cr
TP-7
TP-8
PBF-3
To Trail
TP-6
Merry Cr
To Grand Forks
Schofield Cr
Castlegar sewage lagoon
Pass Cr
PBF-2
TP-5
East Raspberry
TPA-3
TP-18
CP Rail
TP-4
CASTLEGAR
Raspberry
Ferry crossing
Robson
COLUMBIA RIVER
TPA-2
PBF-1
TP-3
TPA-1
TP-2
B.C. Timber, Pulp and Sawmill 0.00 to 4.45
Balfour
TP-1
Hugh Keenleyside Dam
TP-23
Unimproved road relocation 19.87 to 21.05 TP-24
TP-9
Blueberry Creek
TP-10
TP-11
Champion Cr
TP-25
Cemetery
Champion Creek
Fairview
TP-12
Blueberry Cr
TP-13
PBF-4
Jordan Cr
China Creek
China Cr
TP-14
Genelle
TP-15
TP-16
COLUMBIA
Birchbank
Sullivan Cr
Johnson Cr
TP-17 C.P. Rail relocation 27.65 to 32.95
ELEVATION IN METRES
460
450
440
430
420
410
House
Steep slope
Potential long term line of regression, if unprotected
Reservoir impact line
Crest of protective works
Normal operating level El. 424
1
1.5
Potential extent of beaching
Present river level
Toe protection
ELEVATION IN METRES
440
430
420
410
Reservoir impact line
Access berm
Crest of protective works
Existing house and services
Normal operating level El. 424
Present river level
Protective berm and fill

TYPICAL SECTION WITH TOE PROTECTION
Scale: B

TYPICAL SECTION WITH PROTECTIVE BERM AND FILL
Scale: B

Scale: B 1
Scale: A 5
COLUM
SHO
GEN
DATE


Unimproved road relocation 19.87 to 21.05 TP-24
Champion Cr
Jordan Cr
TP-25
Cemetery
Champion Creek
PBF-4
China Creek
TP-14
Genelle
TP-15
TP-16
COLUMBIA 30
RIVER
Birchbank
Sullivan Cr
Johnson Cr
TP-17 C.P. Rail relocation 27.65 to 32.95
Murphy Cr
McNally Cr
Proposed Murphy Creek Project Dam axis (33.70)
Billy Cr
To Trail

LEGEND:

▨	POSSIBLE AREA FOR PROTECTIVE BERM AND FILL, PLACED BEFORE RESERVOIR FILLING
▩	TOE PROTECTION, TO BE PLACED BEFORE RESERVOIR FILLING (RESIDENTIAL AND INDUSTRAIL BUILDINGS, RAILWAY, SEWAGE LAGOON, CEMETERIES AND ROADS)
▤	TOE PROTECTION, TO BE PLACED AFTER RESERVOIR FILLING, IF REQUIRED (ROADS)
5 ●	DISTANCE MEASURED IN KILOMETERES ALONG THE CENTRELINE OF COLUMBIA RIVER, KOOTENAY RIVER AND PASS CREEK
PBF-1	PROTECTIVE BERM AND FILL, LOCATION #1, TO BE PLACED BEFORE RESERVOIR FILLING
TP-1	TOE PROTECTION, LOCATION #1, TO BE PLACED BEFORE RESERVOIR FILLING
TPA-1	TOE PROTECTION, LOCATION #1, TO BE PLACED AFTER RESERVOIR FILLING, IF REQUIRED
ᴧᴧᴧᴧᴧ	EXISTING 'STEEP' SLOPES (STEEPER THAN 1.5H:1V)

Scale: B 10 0 10 20 30 40 Metres

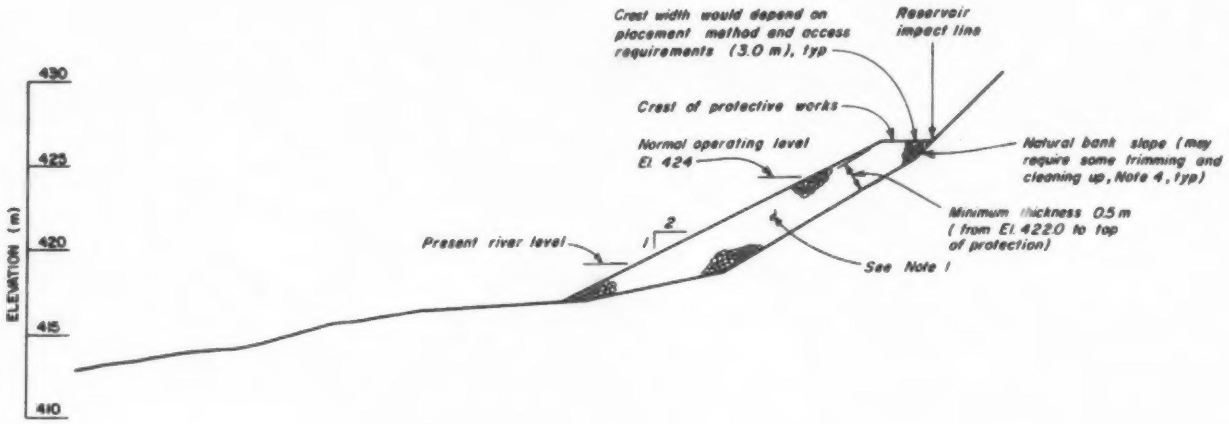
Scale: A 500 0 500 1000 1500 2000 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

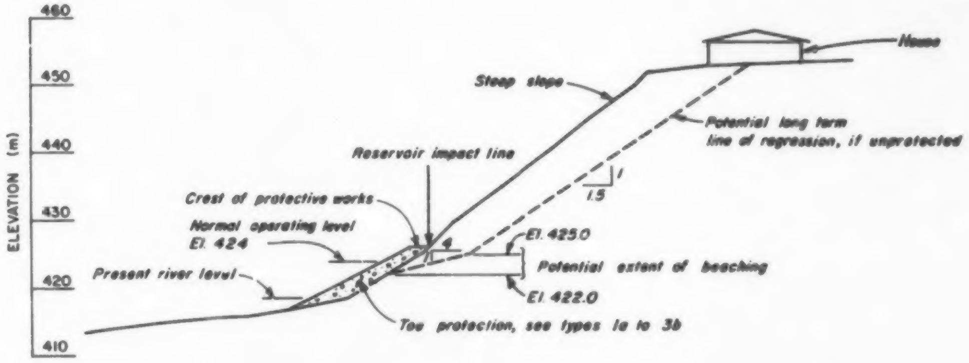
COLUMBIA RIVER - MURPHY CREEK PROJECT

SHORELINE PROTECTION WORKS
GENERAL PLAN

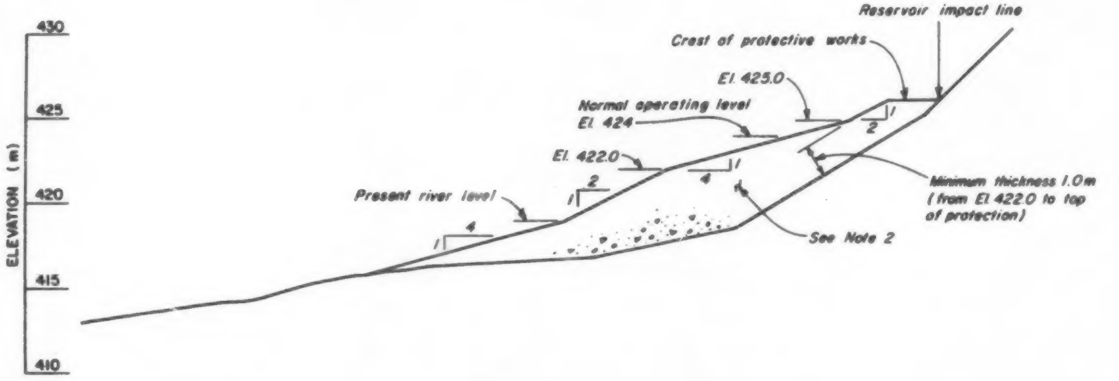
DATE MAY 1983	FIG. 8-2
DWN E	DWG No. 218-C14-U621


Crest width would depend on placement method and access requirements (3.0 m), typ
Reservoir impact line
Crest of protective works
Normal operating level El. 424
Natural bank slope (may require some trimming and cleaning up, Note 4, typ)
Minimum thickness 0.5 m (from El. 422.0 to top of protection)
Present river level
1
2
See Note 1
ELEVATION (m)
430
425
420
415
410

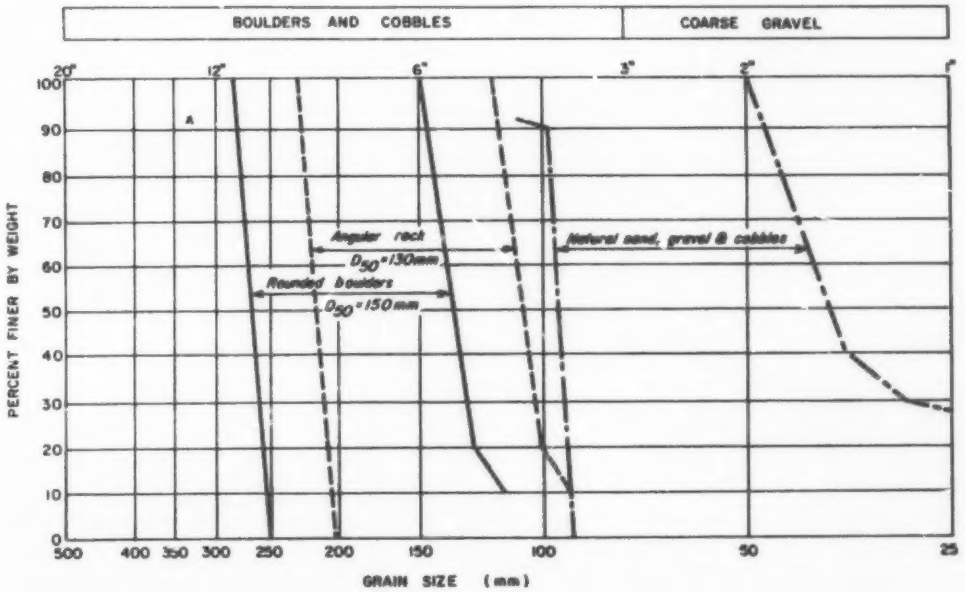
TOE PROTECTION - TYPE 1a & 1b
Scale: A


House
Steep slope
Potential long term line of regression, if unprotected
Reservoir impact line
1
1.5
Crest of protective works
Normal operating level El. 424
El. 425.0
Potential extent of beaching
El. 422.0
Present river level
Toe protection, see types 1a to 3b
ELEVATION (m)
460
450
440
430
420
410

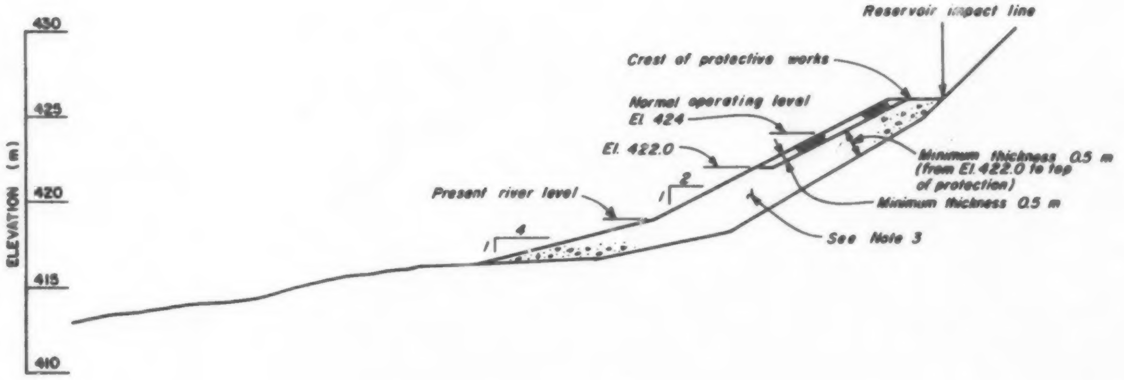
TYPICAL SECTION WITH TOE PROTECTION
Scale: B


Reservoir impact line
Crest of protective works
El. 425.0
Normal operating level El. 424
1
2
El. 422.0
2
4
Present river level
4
Minimum thickness 1.0 m (from El. 422.0 to top of protection)
See Note 2
ELEVATION (m)
430
425
420
415
410

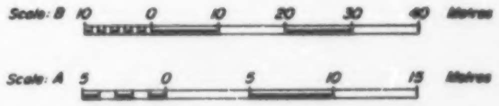
TOE PROTECTION - TYPE 2
Scale: A



PRELIMINARY GRADATIONS


Reservoir impact line
Crest of protective works
Normal operating level El. 424
El. 422.0
1
2
Present river level
1
4
Minimum thickness 0.5 m (from El. 422.0 to top of protection)
Minimum thickness 0.5 m
See Note 3
ELEVATION (m)
430
425
420
415
410

TOE PROTECTION - TYPE 3a & 3b
Scale: A


Scale: B 10 0 10 20 30 40 Metres
Scale: A 5 0 5 10 15 Metres

NOTES:

1. TYPE 1a TOE PROTECTION WOULD CONSIST ENTIRELY OF ANGULAR ROCK.
   TYPE 1b TOE PROTECTION WOULD CONSIST ENTIRELY OF ROUNDED BOULDERS.
2. TYPE 2 TOE PROTECTION WOULD CONSIST ENTIRELY OF NATURAL SAND, GRAVEL AND COBBLES.
3. TYPE 3a TOE PROTECTION WOULD CONSIST OF A COMBINATION OF ANGULAR ROCK OVERLYING NATURAL SAND, GRAVEL AND COBBLES.
   TYPE 3b TOE PROTECTION WOULD CONSIST OF A COMBINATION OF ROUNDED BOULDERS OVERLYING NATURAL SAND, GRAVEL AND COBBLES.
4. THE NEED FOR A FILTER BETWEEN THE EXISTING GROUND SURFACE AND THE PROTECTION WOULD DEPEND ON THE TYPE OF TOE PROTECTION SELECTED AND THE GRADATION OF THE UNDERLYING MATERIAL

LEGEND:

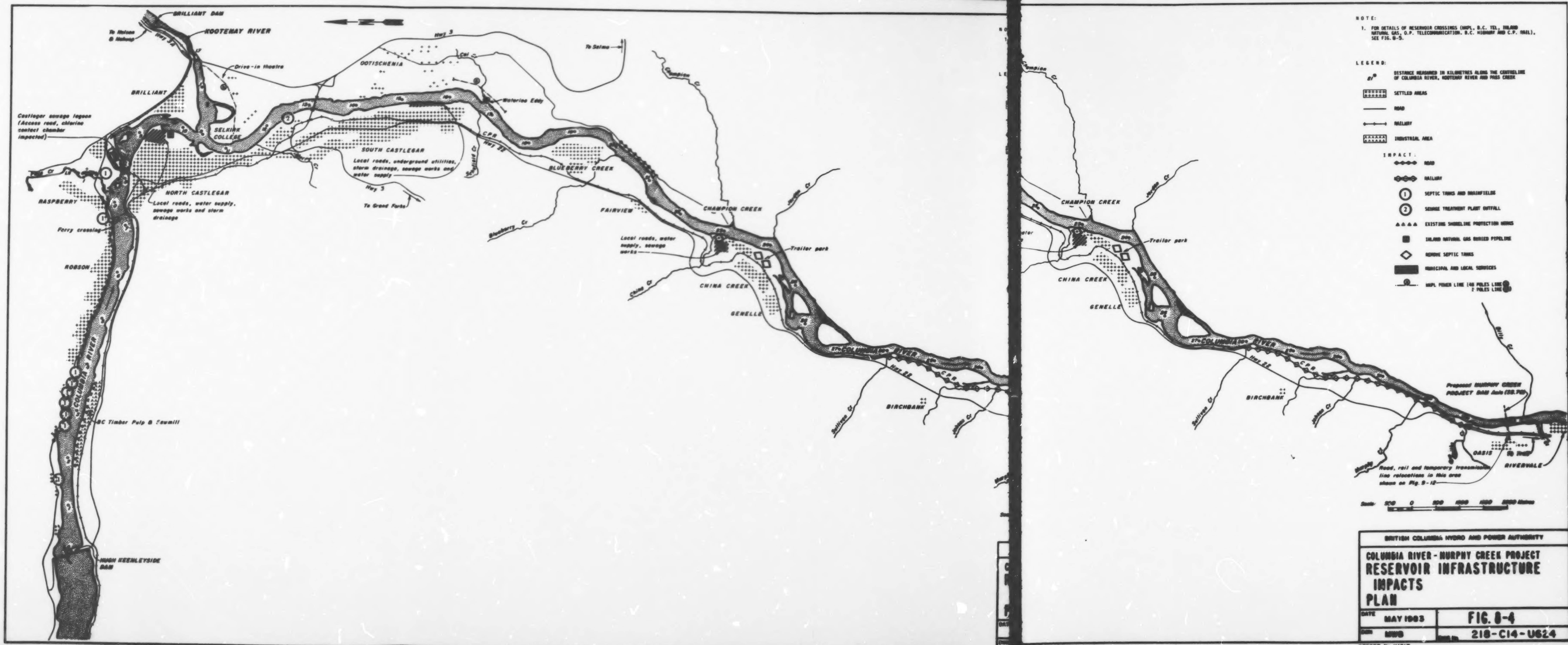
■ ANGULAR ROCK OR ROUNDED BOULDERS
▨ NATURAL SAND, GRAVEL AND COBBLES

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

RESERVOIR SHORELINE PROTECTION WORKS
TYPICAL TOE PROTECTION

DATE MAY 1983	FIG. 8-3	R
DWN J.E.	218-C14-D622	


BRILLIANT DAM
To Nelson & Nelway
Hwy 3A
KOOTENAY RIVER
17
Drive-in theatre
Hwy 3
OOTISCHENIA
Cai Cr
To Salmo
Waterloo Eddy
Champion Cr
BRILLIANT
Castlegar sewage lagoon (Access road, chlorine contact chamber impacted)
SELKIRK COLLEGE
SOUTH CASTLEGAR
Local roads, underground utilities, storm drainage, sewage works and water supply
CPR
Hwy 22
Smallfield Cr
BLUEBERRY CREEK
NORTH CASTLEGAR
Local roads, water supply, sewage works and storm drainage
Hwy 3
To Grand Forks
RASPBERRY
Ferry crossing
ROBSON
FAIRVIEW
Blueberry Cr
CHAMPION CREEK
Jordan Cr
Local roads, water supply, sewage works
Trailer park
China Cr
CHINA CREEK
GENELLE
COLUMBIA RIVER
Hwy 22
CPR
BIRCHBANK
Sullivan Cr
Johnson Cr
BC Timber Pulp & Sawmill
HUGH KEENLEYSIDE DAM
Proposed MURPHY CREEK PROJECT DAM Axis (33.7 km)
Bills Cr
OASIS
To Trail
RIVERVALE
Road, rail and temporary transmission line relocations in this area shown on Fig. 9-12
Scale: 500 0 500 1000 1500 2000 Metres

NOTE:

1. FOR DETAILS OF RESERVOIR CROSSINGS (WKPL, B.C. TEL, INLAND NATURAL GAS, O.P. TELECOMMUNICATION, B.C. HIGHWAY AND C.P. RAIL), SEE FIG. 8-5.

LEGEND:
DISTANCE MEASURED IN KILOMETRES ALONG THE CENTRELINE OF COLUMBIA RIVER, KOOTENAY RIVER AND PASS CREEK
SETTLED AREAS
ROAD
RAILWAY
INDUSTRIAL AREA
IMPACT:
ROAD
RAILWAY
① SEPTIC TANKS AND DRAINFIELDS
② SEWAGE TREATMENT PLANT OUTFALL
▲▲▲▲ EXISTING SHORELINE PROTECTION WORKS
INLAND NATURAL GAS BURIED PIPELINE
◇ REMOVE SEPTIC TANKS
MUNICIPAL AND LOCAL SERVICES
WKPL POWER LINE (40 POLES LINE / 2 POLES LINE)

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT
RESERVOIR INFRASTRUCTURE IMPACTS
PLAN

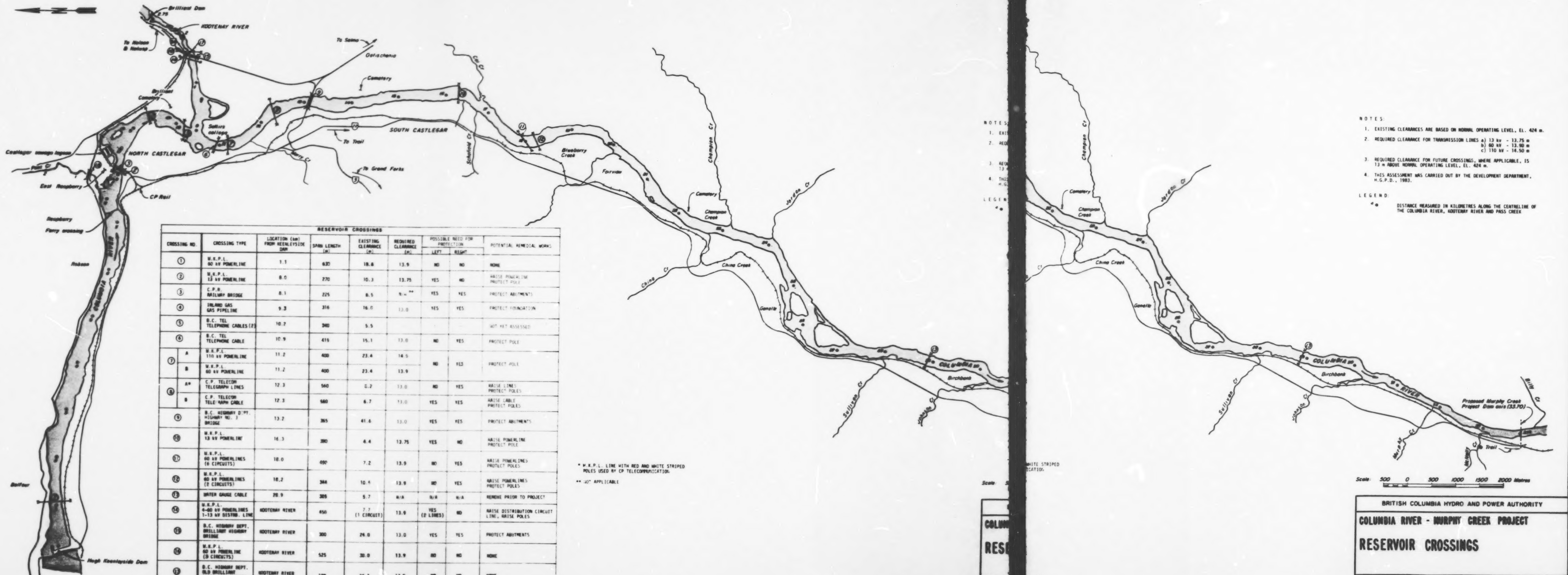
DATE MAY 1983

FIG. 8-4

DWN MWB

Dwg. No. 218-C14-U624

REPORT No H1617


N
Brilliant Dam
2.75
KOOTENAY RIVER
To Nelson
& Nelsop
To Salmo
Ootischenia
Cemetery
Brilliant
Cemetery
Selkirk college
SOUTH CASTLEGAR
To Trail
To Grand Forks
Castlegar sewage lagoon
NORTH CASTLEGAR
Pass Cr
East Raspberry
CP Rail
Raspberry
Ferry crossing
Robson
COLUMBIA RIVER
Balfour
Hugh Keenleyside Dam
Cai Cr
Schofield Cr
Blueberry Creek
Fairview
Champion Cr
Cemetery
Champion Creek
Jordan Cr
China Cr
China Creek
Genelle
COLUMBIA RIVER
Birchbank
Sullivan Cr
Johnson Cr
Proposed Murphy Creek Project Dam axis (33.70)
Murphy Cr
McNally Cr
Trail
Bills Cr
Scale: 500 0 500 1000 1500 2000 Metres

CROSSING NO.		CROSSING TYPE	LOCATION (km) FROM KEENLEYSIDE DAM	SPAN LENGTH (m)	EXISTING CLEARANCE (m)	REQUIRED CLEARANCE (m)	POSSIBLE NEED FOR PROTECTION		POTENTIAL REMEDIAL WORKS
							LEFT	RIGHT	
①		W.K.P.L. 60 kV POWERLINE	1.1	630	18.8	13.9	NO	NO	NONE
②		W.K.P.L. 13 kV POWERLINE	8.0	270	10.3	13.75	YES	NO	RAISE POWERLINE PROTECT POLE
③		C.P.R. RAILWAY BRIDGE	8.1	225	8.5	N/A **	YES	YES	PROTECT ABUTMENTS
④		INLAND GAS GAS PIPELINE	9.3	316	16.0	13.0	YES	YES	PROTECT FOUNDATION
⑤		B.C. TEL TELEPHONE CABLES (2)	10.2	340	5.5		-	-	NOT YET ASSESSED
⑥		B.C. TEL TELEPHONE CABLE	10.9	415	15.1	13.0	NO	YES	PROTECT POLE
⑦	A	W.K.P.L. 110 kV POWERLINE	11.2	400	23.4	14.5	NO	YES	PROTECT POLE
	B	W.K.P.L. 60 kV POWERLINE	11.2	400	23.4	13.9			
⑧	A*	C.P. TELECOM TELEGRAPH LINES	12.3	560	8.2	13.0	NO	YES	RAISE LINES PROTECT POLES
	B	C.P. TELECOM TELEGRAPH CABLE	12.3	560	6.7	13.0	YES	YES	RAISE CABLE PROTECT POLES
⑨		B.C. HIGHWAY DEPT. HIGHWAY NO. 3 BRIDGE	13.2	365	41.6	13.0	YES	YES	PROTECT ABUTMENTS
⑩		W.K.P.L. 13 kV POWERLINE	16.3	380	4.4	13.75	YES	NO	RAISE POWERLINE PROTECT POLE
⑪		W.K.P.L. 60 kV POWERLINES (6 CIRCUITS)	18.0	490	7.2	13.9	NO	YES	RAISE POWERLINES PROTECT POLES
⑫		W.K.P.L. 60 kV POWERLINES (2 CIRCUITS)	18.2	344	10.4	13.9	NO	YES	RAISE POWERLINES PROTECT POLES
⑬		WATER GAUGE CABLE	28.9	305	5.7	N/A	N/A	N/A	REMOVE PRIOR TO PROJECT
⑭		W.K.P.L. 4-60 kV POWERLINES 1-13 kV DISTRB. LINE	KOOTENAY RIVER	450	7.7 (1 CIRCUIT)	13.9	YES (2 LINES)	NO	RAISE DISTRIBUTION CIRCUIT LINE, RAISE POLES
⑮		B.C. HIGHWAY DEPT. BRILLIANT HIGHWAY BRIDGE	KOOTENAY RIVER	300	24.0	13.0	YES	YES	PROTECT ABUTMENTS
⑯		W.K.P.L. 60 kV POWERLINE (3 CIRCUITS)	KOOTENAY RIVER	525	30.0	13.9	NO	NO	NONE
⑰		B.C. HIGHWAY DEPT. OLD BRILLIANT SUSPENSION BRIDGE	KOOTENAY RIVER	120	16.1	13.0	NO	NO	NONE
⑱		B.C. HIGHWAY DEPT. HIGHWAY BRIDGE	PASS CREEK	45	4.2	N/A	NO	NO	NONE

* W.K.P.L. LINE WITH RED AND WHITE STRIPED POLES USED BY CP TELECOMMUNICATION.

** NOT APPLICABLE

NOTES:

1. EXISTING CLEARANCES ARE BASED ON NORMAL OPERATING LEVEL, EL. 424 m.
2. REQUIRED CLEARANCE FOR TRANSMISSION LINES a) 13 kV - 13.75 m
   b) 60 kV - 13.90 m
   c) 110 kV - 14.50 m
3. REQUIRED CLEARANCE FOR FUTURE CROSSINGS, WHERE APPLICABLE, IS 13 m ABOVE NORMAL OPERATING LEVEL, EL. 424 m.
4. THIS ASSESSMENT WAS CARRIED OUT BY THE DEVELOPMENT DEPARTMENT, H.G.P.D., 1983.

LEGEND:

●● DISTANCE MEASURED IN KILOMETRES ALONG THE CENTRELINE OF THE COLUMBIA RIVER, KOOTENAY RIVER AND PASS CREEK

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

RESERVOIR CROSSINGS

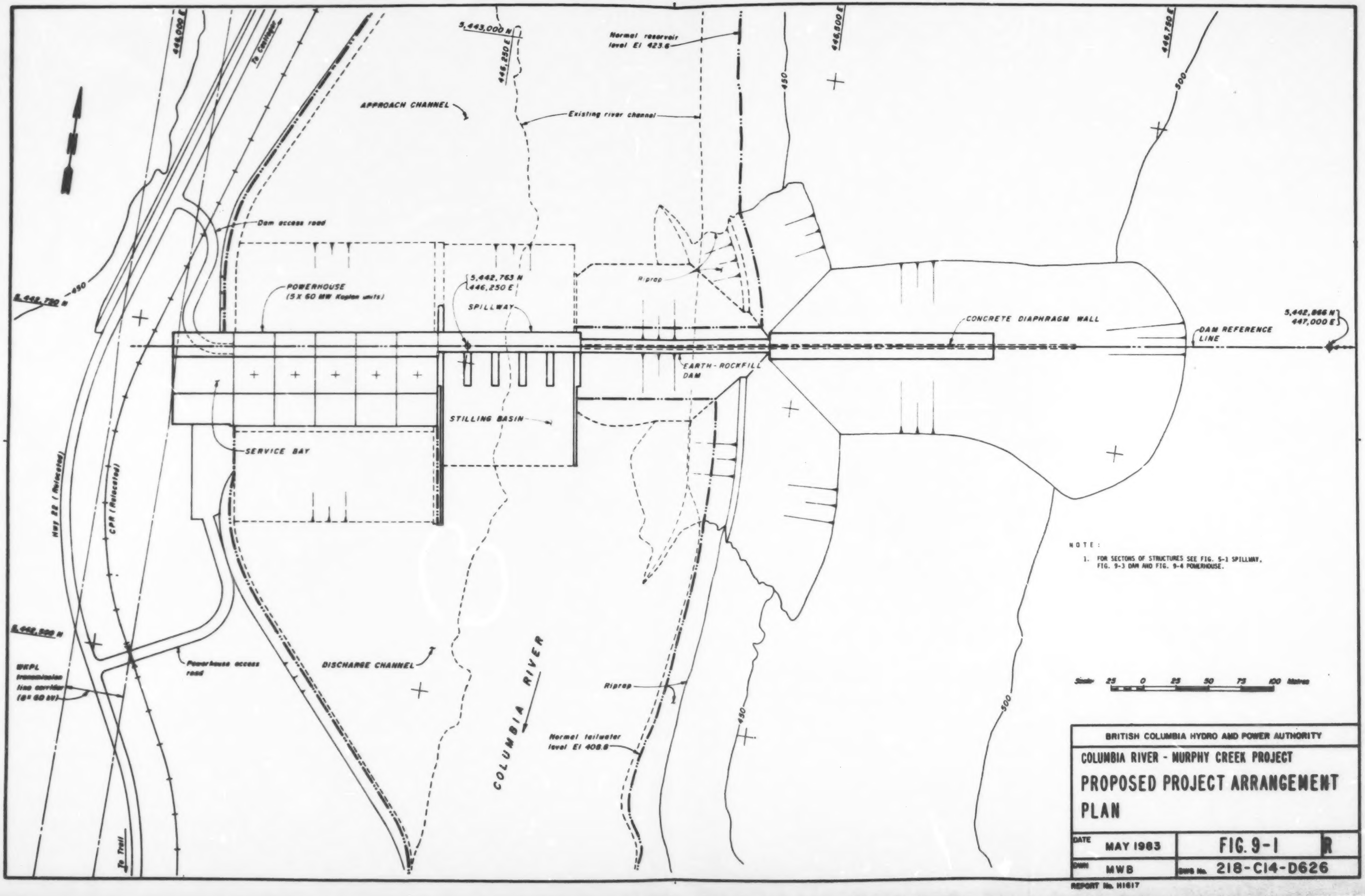
DATE MAY 1983

FIG. 8-5

DWN MWB

DWG No. 218-C14-U625

NOTES
1. EXI
2. REQ
3. REQ
13
4. THI
H.G
LEGEN
HITE STRIPED
CATION
Scale: 5
COLUM
RESE
DATE M
DWN M


446,000 E
To Castlegar
5,443,000 N
446,250 E
Normal reservoir level El 423.6
446,500 E
446,750 E
APPROACH CHANNEL
Existing river channel
450
500
Dam access road
5,442,750 N
POWERHOUSE (5 X 60 MW Kaplan units)
5,442,763 N 446,250 E
SPILLWAY
Riprap
CONCRETE DIAPHRAGM WALL
5,442,866 N 447,000 E
DAM REFERENCE LINE
EARTH-ROCKFILL DAM
STILLING BASIN
SERVICE BAY
Hwy 22 (Relocated)
CPR (Relocated)
5,442,500 N
WKPL transmission line corridor (8" 60 kV)
Powerhouse access road
DISCHARGE CHANNEL
COLUMBIA RIVER
Riprap
Normal tailwater level El 408.8
To Trail
Scale 25 0 25 50 75 100 Metres

NOTE:

1. FOR SECTONS OF STRUCTURES SEE FIG. 5-1 SPILLWAY, FIG. 9-3 DAM AND FIG. 9-4 POWERHOUSE.

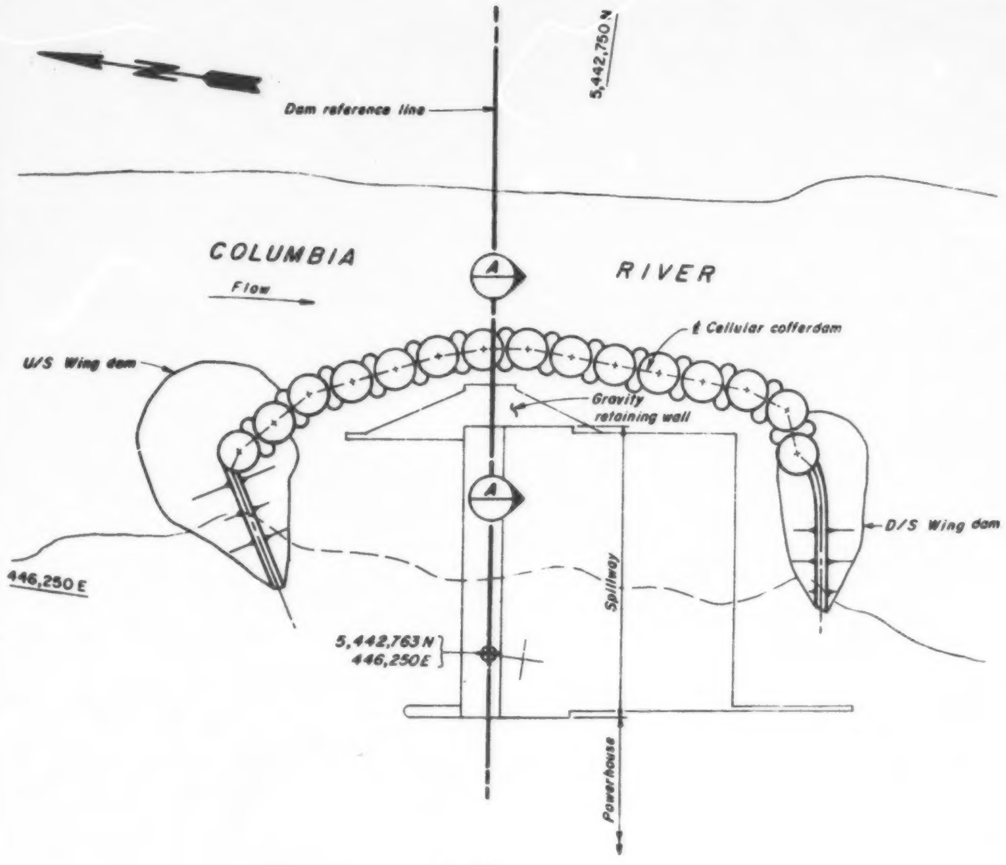
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

PROPOSED PROJECT ARRANGEMENT PLAN

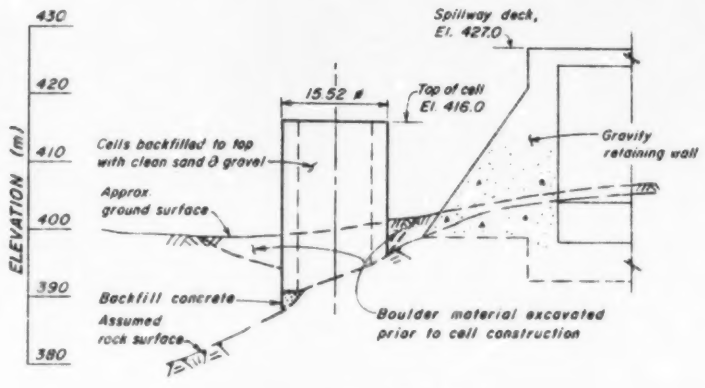
DATE	MAY 1983	FIG. 9-1	R
DWN	MWB	DWG No. 218-C14-D626	

REPORT No. H1617



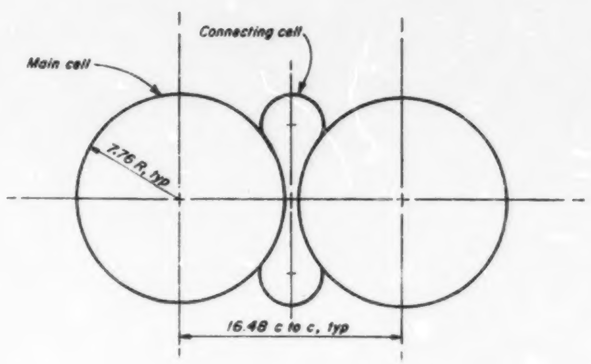
5,442,750 N
Dam reference line
COLUMBIA
RIVER
Flow
A
¢ Cellular cofferdam
U/S Wing dam
Gravity retaining wall
A
D/S Wing dam
446,250 E
Spillway
5,442,763 N
446,250 E
Powerhouse

PLAN
Scale A



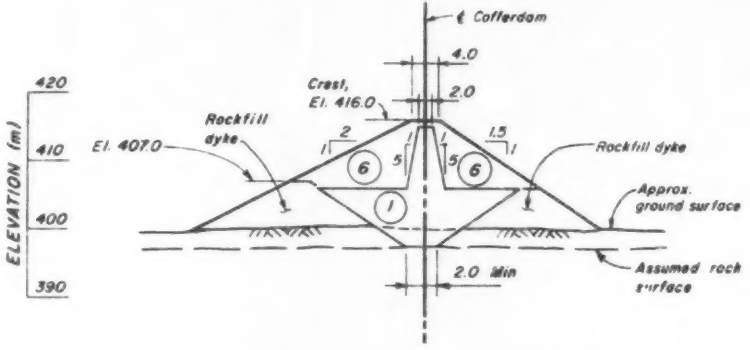
430
420
410
400
390
380
ELEVATION (m)
Spillway deck, El. 427.0
15.52 ±
Top of cell El. 416.0
Cells backfilled to top with clean sand & gravel
Gravity retaining wall
Approx. ground surface
Backfill concrete
Assumed rock surface
Boulder material excavated prior to cell construction

SECTION A
Scale B



Connecting cell
Main cell
7.76 R, typ
16.48 c to c, typ

TYPICAL CELL LAYOUT
Scale C



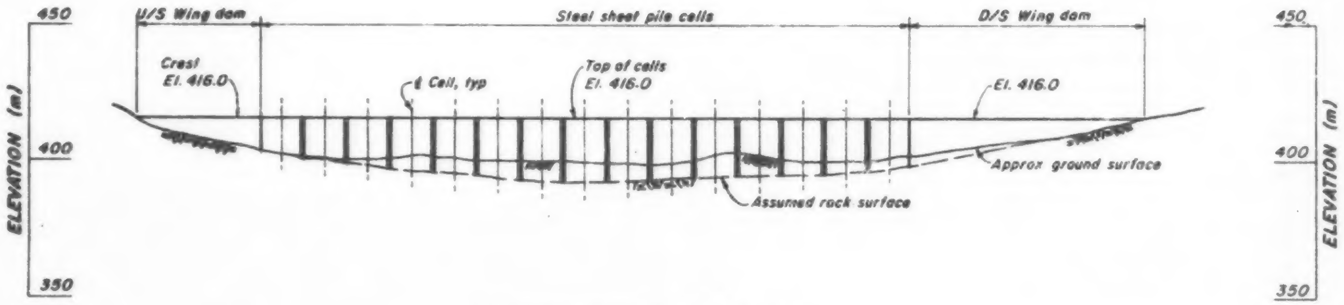
¢ Cofferdam
4.0
2.0
Crest, El. 416.0
420
410
400
390
ELEVATION (m)
El. 407.0
Rockfill dyke
2
1
5
6
1
5
6
1.5
1
Rockfill dyke
Approx. ground surface
2.0 Min
Assumed rock surface

TYPICAL WING DAM SECTION
Scale B

LEGEND

① Till core

⑥ Rockfill



450
400
350
ELEVATION (m)
U/S Wing dam
Steel sheet pile cells
D/S Wing dam
Crest El. 416.0
¢ Cell, typ
Top of cells El. 416.0
El. 416.0
Approx ground surface
Assumed rock surface
450
400
350
ELEVATION (m)

SECTION ALONG ¢ COFFERDAM
(DEVELOPED)
Scale A

Scale C: 5 0 5 10 15 20 Metres
Scale B: 10 0 10 20 30 40 Metres
Scale A: 25 0 25 50 75 100 Metres

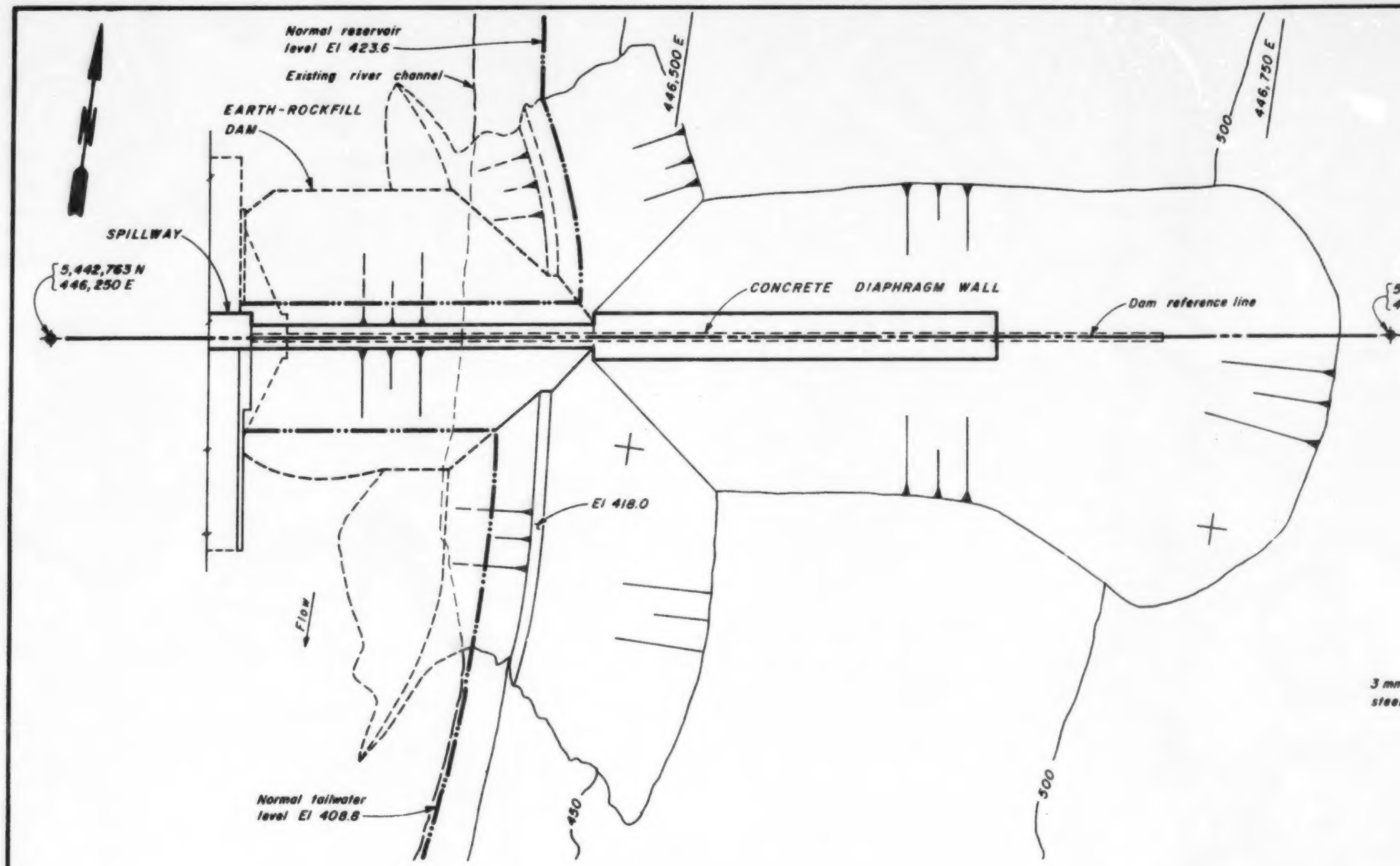
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

COFFERDAM ARRANGEMENT
PLAN & SECTIONS

DATE MAY 1983	FIG. 9-2
DWN NO	DWG No. 218-C14-D628

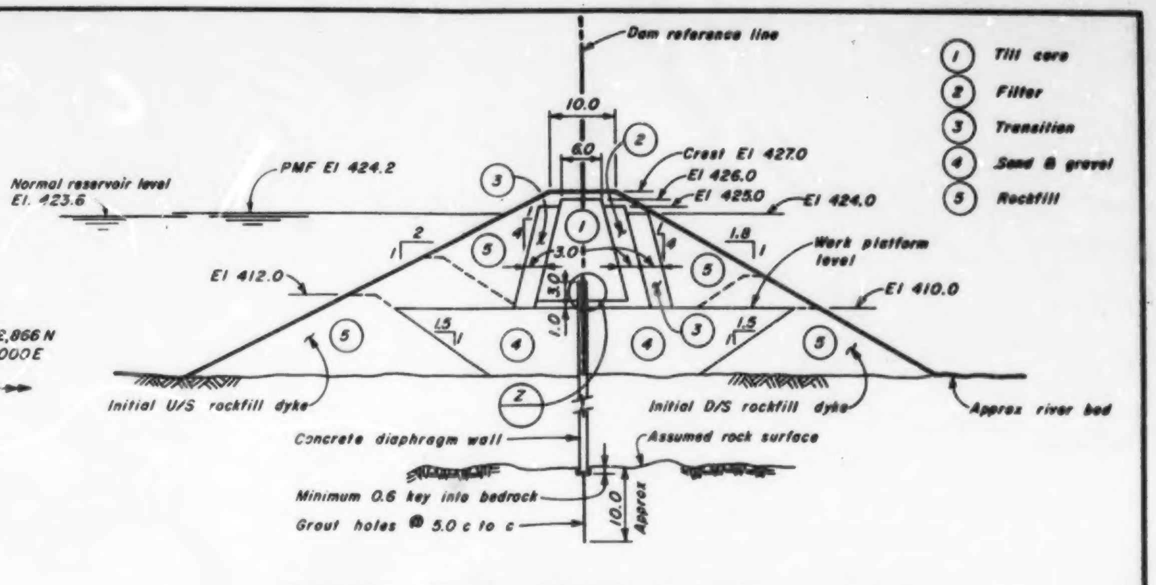
REPORT No. H1617



N
Normal reservoir level El 423.6
Existing river channel
EARTH-ROCKFILL DAM
SPILLWAY
5,442,763 N
446,250 E
CONCRETE DIAPHRAGM WALL
Dam reference line
5,442,866 N
447,000E
446,500 E
446,750 E
500
El 418.0
Flow
Normal tailwater level El 408.8
450
500

EARTH-ROCKFILL DAM - PLAN

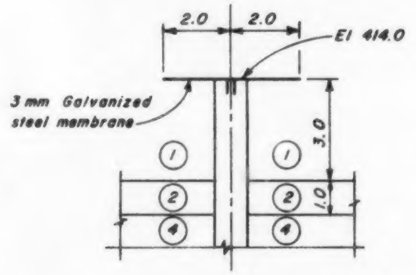
Scale : A



Dam reference line
10.0
6.0
Normal reservoir level El. 423.6
PMF El 424.2
Crest El 427.0
El 426.0
El 425.0
El 424.0
2
1.8
Work platform level
El 412.0
3.0
3.0
1.0
1.5
1.5
El 410.0
Initial U/S rockfill dyke
Initial D/S rockfill dyke
Approx river bed
Concrete diaphragm wall
Assumed rock surface
Minimum 0.6 key into bedrock
Grout holes @ 5.0 c to c
10.0 Approx
① Till core
② Filter
③ Transition
④ Sand & gravel
⑤ Rockfill

TYPICAL EARTH - ROCKFILL DAM SECTION

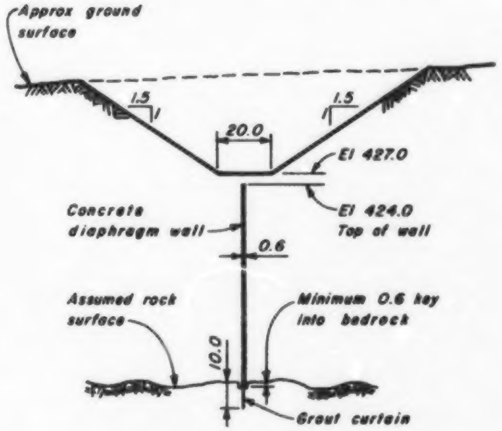
Scale: B



2.0
2.0
El 414.0
3 mm Galvanized steel membrane
3.0
1.0
①
②
④

DETAIL Ⓩ

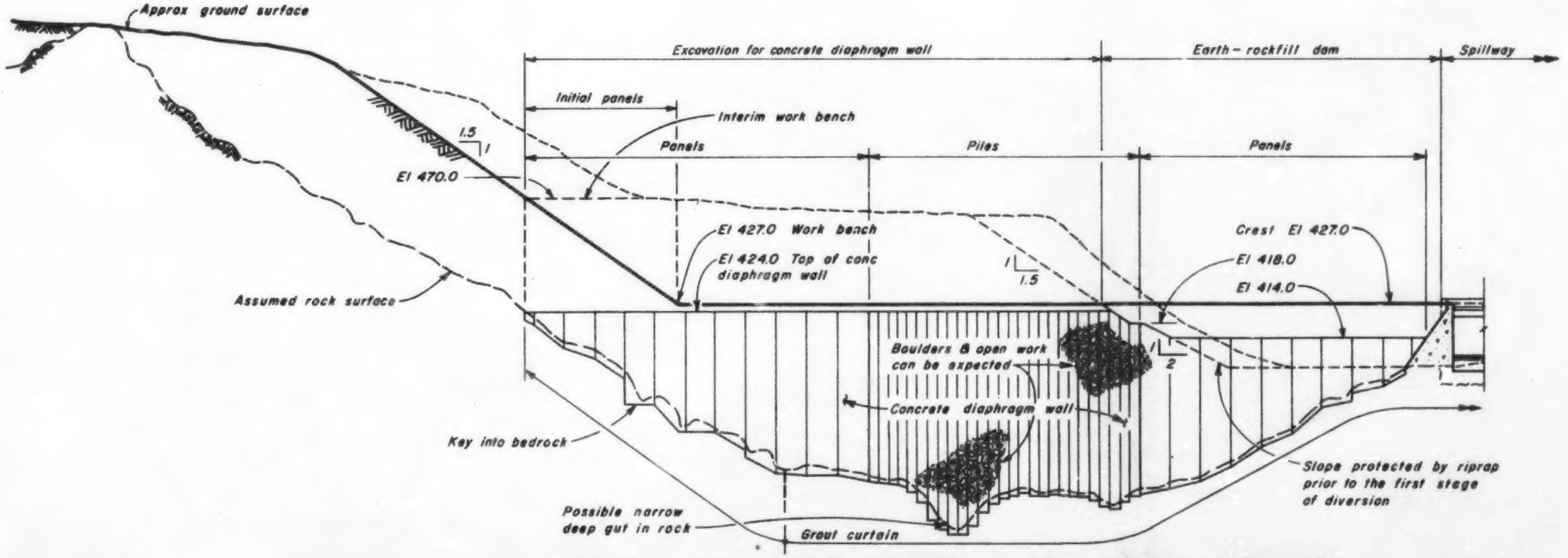
NTS



Approx ground surface
1.5
20.0
1.5
El 427.0
El 424.0 Top of wall
Concrete diaphragm wall
0.6
Assumed rock surface
Minimum 0.6 key into bedrock
10.0
Grout curtain

CONCRETE DIAPHRAGM WALL SECTION - LEFT BANK

NTS



Approx ground surface
Excavation for concrete diaphragm wall
Earth-rockfill dam
Spillway
Initial panels
Interim work bench
1.5
Panels
Piles
Panels
El 470.0
El 427.0 Work bench
El 424.0 Top of conc diaphragm wall
1.5
Crest El 427.0
El 418.0
El 414.0
2
Assumed rock surface
Boulders & open work can be expected
Concrete diaphragm wall
Key into bedrock
Slope protected by riprap prior to the first stage of diversion
Possible narrow deep gut in rock
Grout curtain

SECTION ALONG DAM REFERENCE LINE (LOOKING DOWNSTREAM)

Scale : A

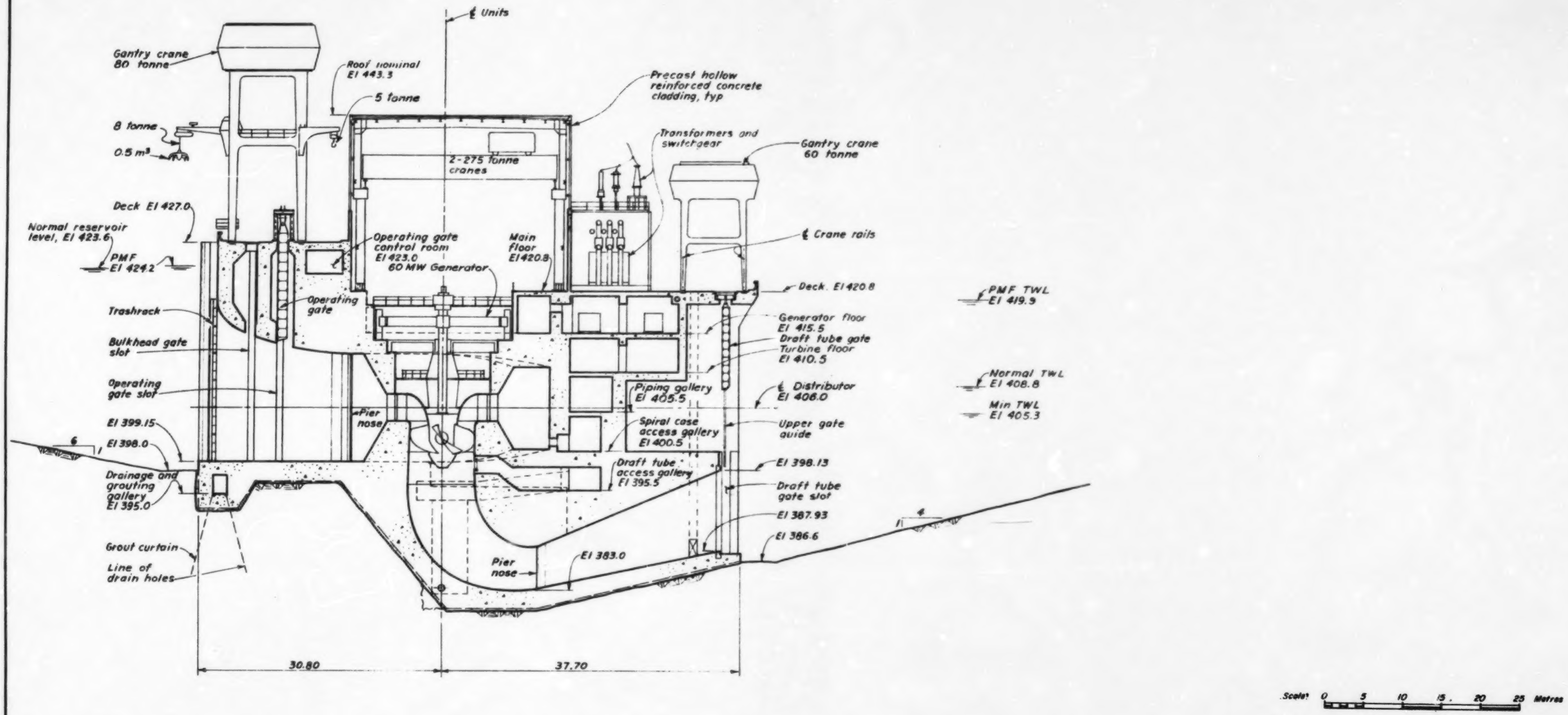
Scale: B 10 0 10 20 30 40 Metres
Scale: A 25 0 25 50 75 100 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER-MURPHY CREEK PROJECT

EARTH-ROCKFILL DAM
PLAN, SECTIONS & DETAIL

DATE MAY 1983	FIG. 9-3
DWN SMD	DWG No. 218-C14-D629


℄ Units
Gantry crane 80 tonne
Roof nominal El 443.3
5 tonne
Precast hollow reinforced concrete cladding, typ
8 tonne
0.5 m³
2-275 tonne cranes
Transformers and switchgear
Gantry crane 60 tonne
Deck El 427.0
Normal reservoir level, El 423.6
PMF El 424.2
Operating gate control room El 423.0
60 MW Generator
Main floor El 420.8
℄ Crane rails
Deck El 420.8
PMF TWL El 419.9
Trashrack
Operating gate
Generator floor El 415.5
Draft tube gate
Turbine floor El 410.5
Bulkhead gate slot
Operating gate slot
Piping gallery El 405.5
℄ Distributor El 406.0
Normal TWL El 408.8
Min TWL El 405.3
Pier nose
El 399.15
El 398.0
6
1
Spiral case access gallery El 400.5
Upper gate guide
Drainage and grouting gallery El 395.0
Draft tube access gallery El 395.5
El 398.13
Draft tube gate slot
El 387.93
El 386.6
4
1
Grout curtain
Line of drain holes
Pier nose
El 383.0
30.80
37.70
Scale 0 5 10 15 20 25 Metres

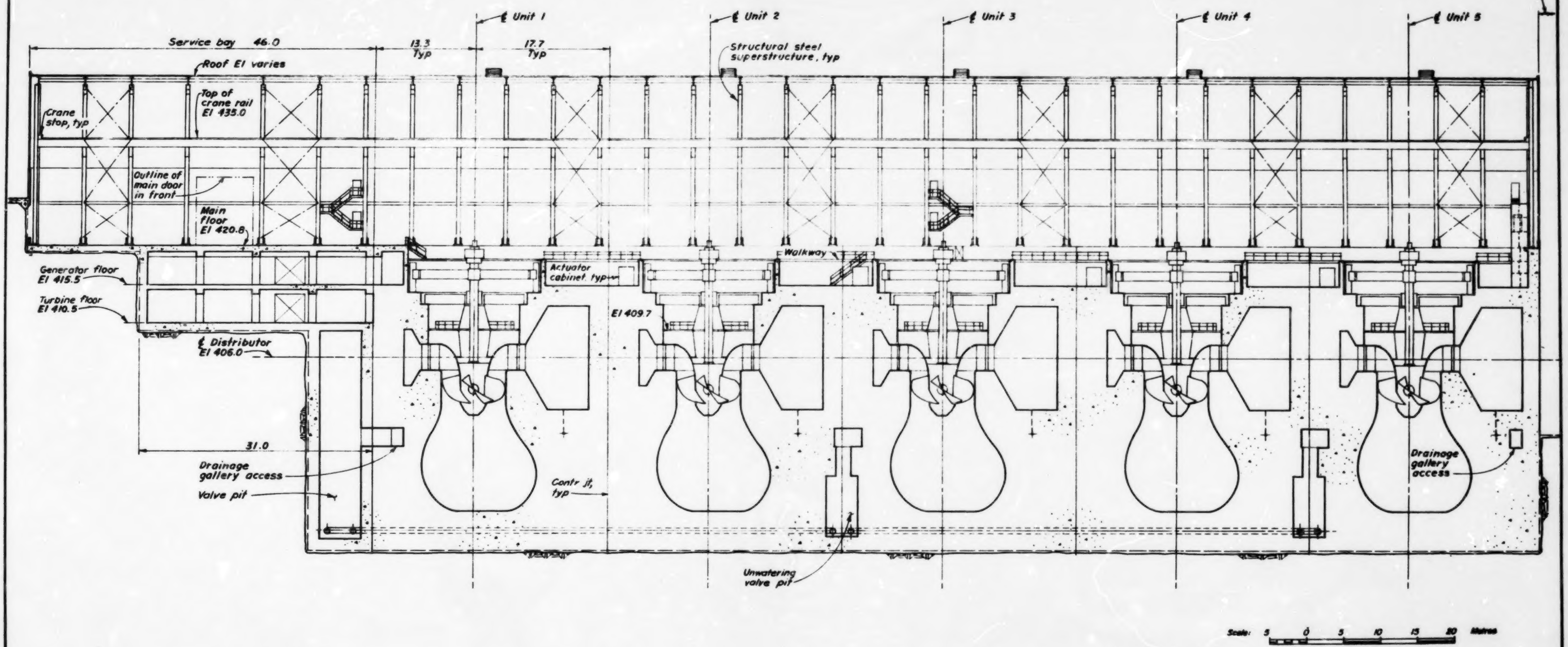
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

POWERHOUSE
TYPICAL CROSS SECTION

DATE	MAY 1983	FIG. 9-4
DWN	MWB	DWG No. 218-C14-D631

REPORT No. H1617

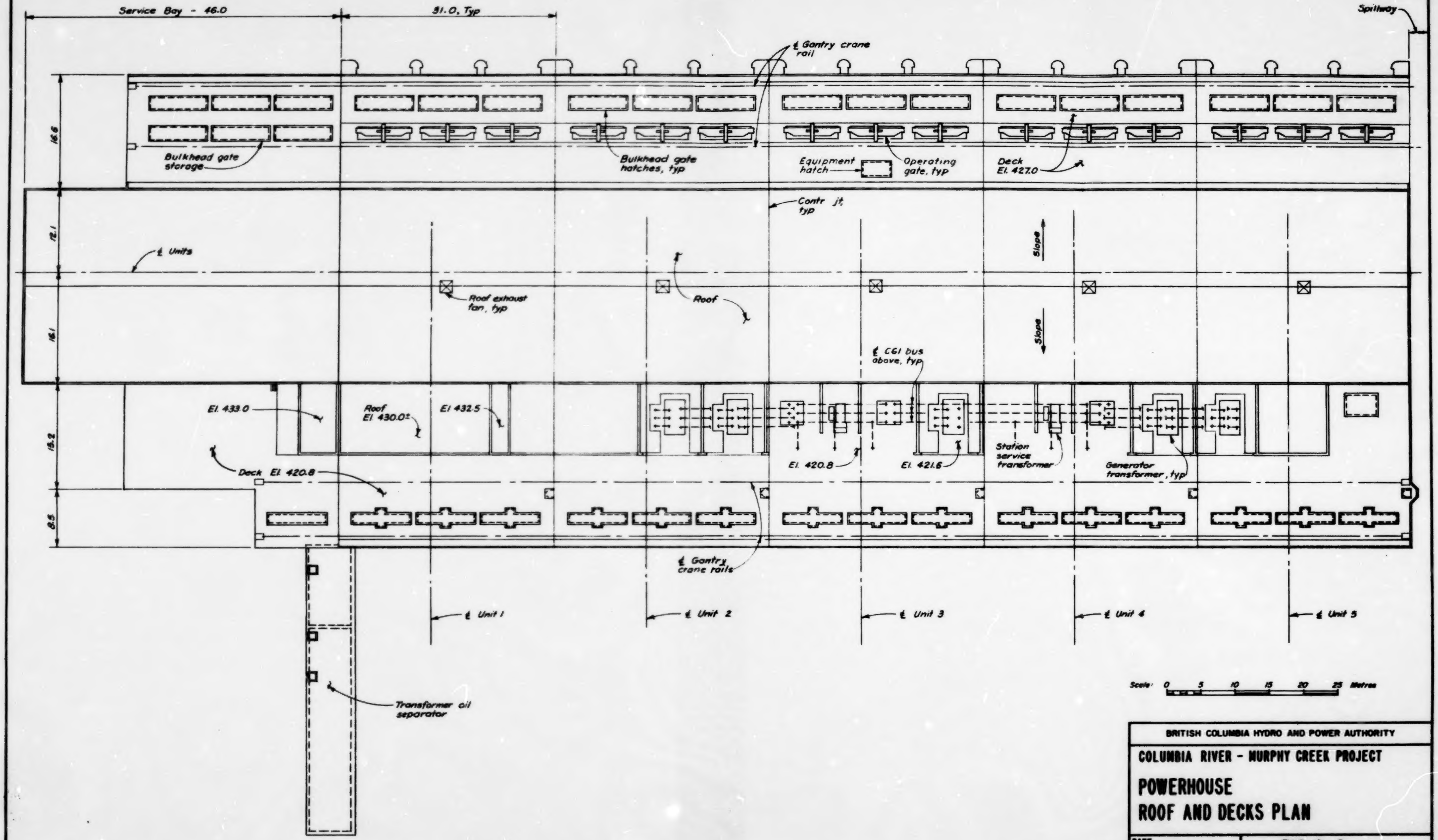

Spillway
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
Service bay 46.0
13.3 Typ
17.7 Typ
Structural steel superstructure, typ
Roof El varies
Top of crane rail El 435.0
Crane stop, typ
Outline of main door in front
Main floor El 420.8
Walkway
Actuator cabinet, typ
Generator floor El 415.5
Turbine floor El 410.5
El 409.7
℄ Distributor El 406.0
31.0
Drainage gallery access
Valve pit
Contr jt, typ
Unwatering valve pit
Drainage gallery access
Scale: 5 0 5 10 15 20 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

POWERHOUSE
LONGITUDINAL SECTION

DATE	MAY 1983		FIG. 9-5
DWN	MWB	DWG No.	218-C14-D632


Service Bay - 46.0
31.0, Typ
Spillway
℄ Gantry crane rail
16.6
12.1
16.1
15.2
8.5
Bulkhead gate storage
Bulkhead gate hatches, typ
Equipment hatch
Operating gate, typ
Deck El. 427.0
Contr jt, typ
℄ Units
Roof exhaust fan, typ
Roof
Slope
Slope
℄ CGI bus above, typ
El. 433.0
Roof El 430.0±
El 432.5
Deck El. 420.8
El. 420.8
El. 421.6
Station service transformer
Generator transformer, typ
℄ Gantry crane rails
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
Transformer oil separator
Scale: 0 5 10 15 20 25 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

POWERHOUSE
ROOF AND DECKS PLAN

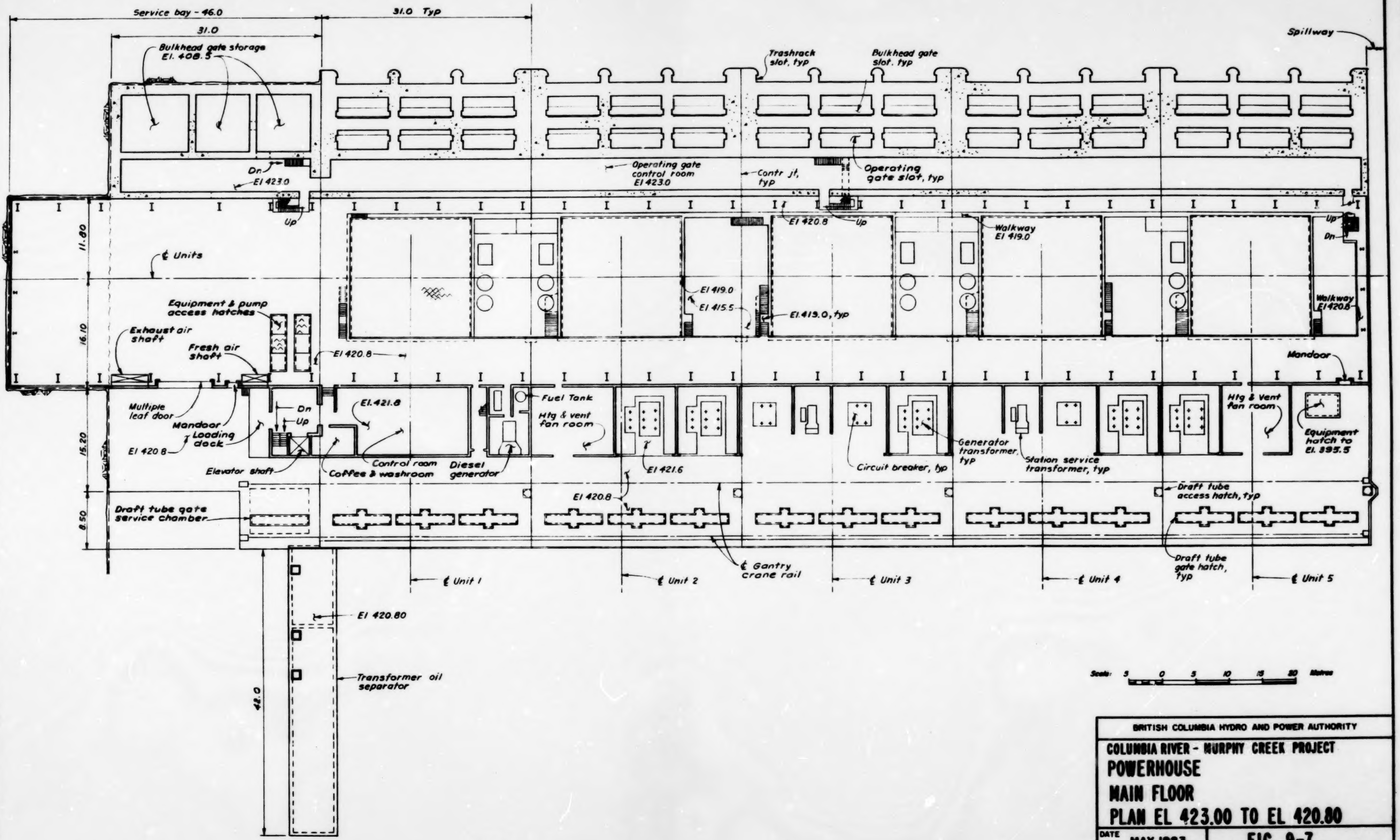
DATE MAY 1983

FIG. 9-6

DWN SMD

DWG No. 218-C14-D633

REPORT No. H1617


Service bay - 46.0
31.0
31.0 Typ
Bulkhead gate storage El. 408.5
Spillway
Trashrack slot, typ
Bulkhead gate slot, typ
Dn
El 423.0
Operating gate control room El 423.0
Contr jt, typ
Operating gate slot, typ
Up
El 420.8
Up
Walkway El 419.0
Up
Dn
11.80
16.10
15.20
8.50
42.0
℄ Units
Equipment & pump access hatches
Exhaust air shaft
Fresh air shaft
El 420.8
El 419.0
El 415.5
El 419.0, typ
Walkway El 420.8
Mandoor
Multiple leaf door
Mandoor
Loading dock
El 420.8
Dn
Up
El. 421.8
Fuel Tank
Htg & vent fan room
Htg & vent fan room
Equipment hatch to El. 395.5
Elevator shaft
Coffee & washroom
Control room
Diesel generator
El 421.6
Generator transformer, typ
Station service transformer, typ
Circuit breaker, typ
Draft tube access hatch, typ
El 420.8
Draft tube gate service chamber
Draft tube gate hatch, typ
℄ Gantry crane rail
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
El 420.80
Transformer oil separator
Scale: 5 0 5 10 15 20 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

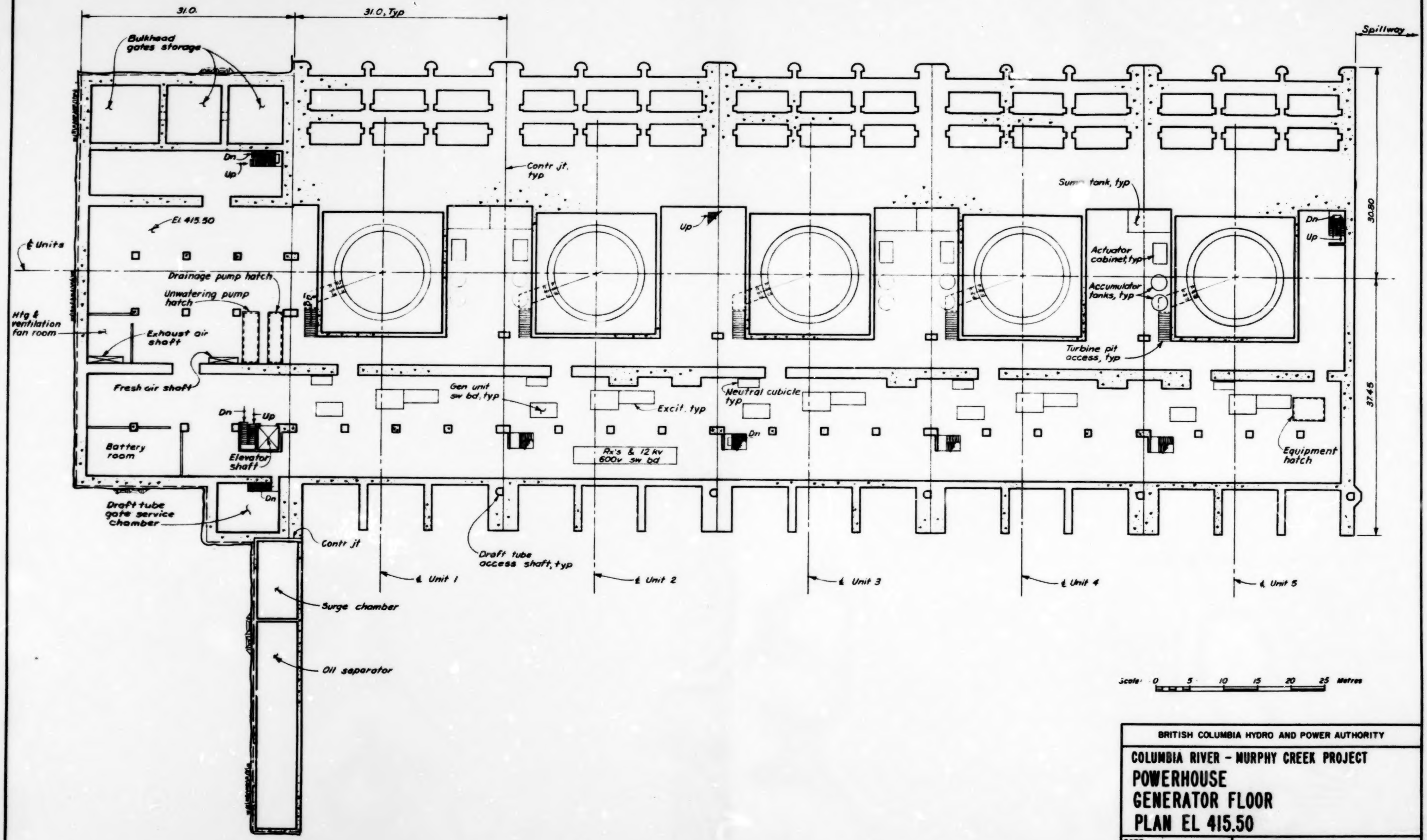
POWERHOUSE

MAIN FLOOR

PLAN EL 423.00 TO EL 420.80

DATE	MAY 1983		FIG. 9-7
DRN	MWB	DWG No.	218-C14-D634

REPORT No. H1617


31.0
31.0, Typ
Spillway
Bulkhead gates storage
Dn
Up
Contr jt, typ
Sump tank, typ
El 415.50
℄ Units
Up
Dn
Up
Actuator cabinet, typ
Accumulator tanks, typ
Drainage pump hatch
Unwatering pump hatch
Htg & ventilation fan room
Exhaust air shaft
Turbine pit access, typ
30.80
37.45
Fresh air shaft
Gen unit sw bd, typ
Neutral cubicle typ
Excit. typ
Dn
Up
Dn
Battery room
Elevator shaft
Rx's & 12 kv 600v sw bd
Equipment hatch
Draft tube gate service chamber
Dn
Contr jt
Draft tube access shaft, typ
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
Surge chamber
Oil separator
Scale: 0 5 10 15 20 25 Metres

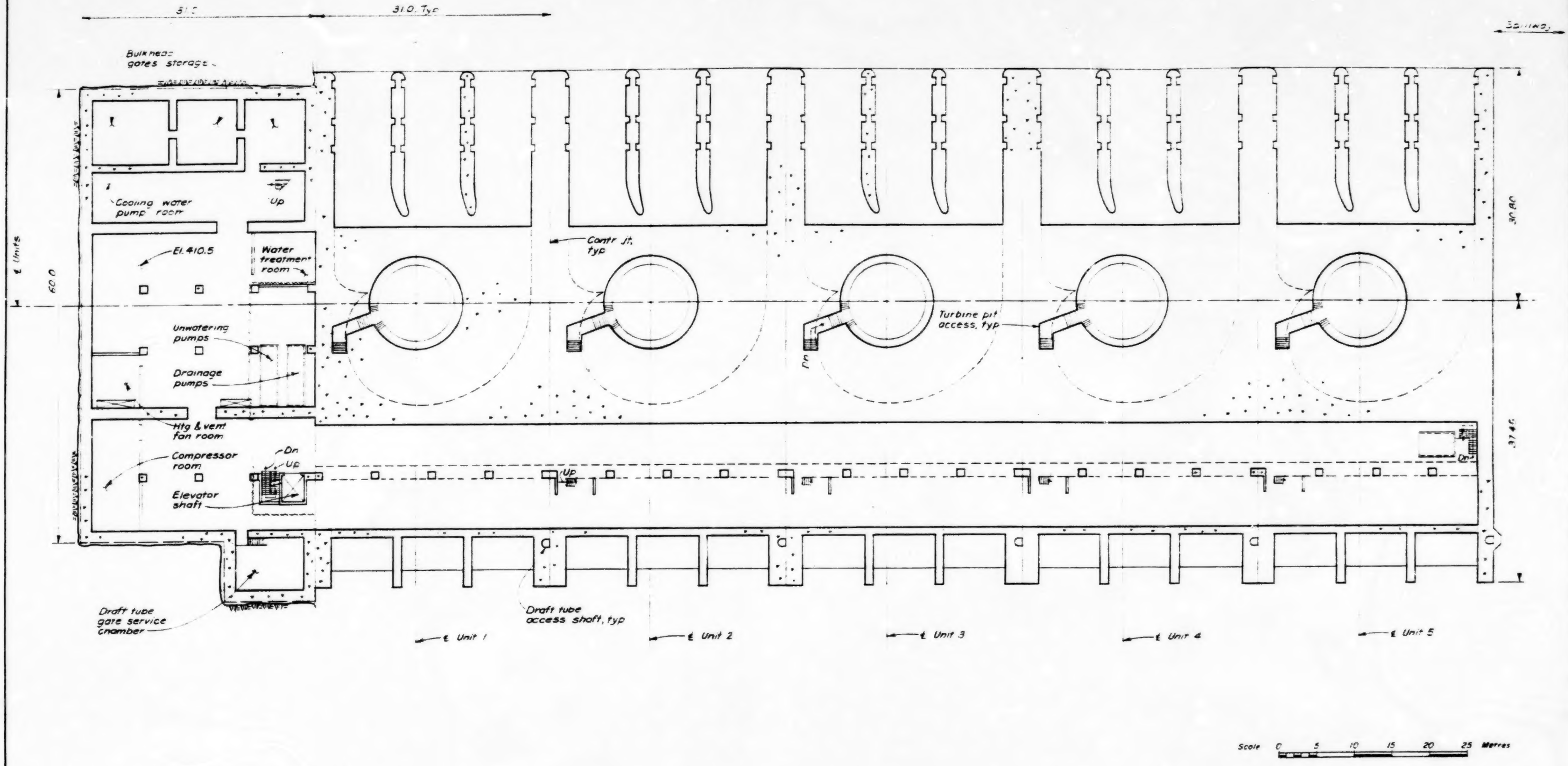
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER – MURPHY CREEK PROJECT

POWERHOUSE
GENERATOR FLOOR
PLAN EL 415.50

DATE MAY 1983	FIG. 9-8
DWN SMD	DWG No. 218-C14-D635

REPORT No. H1617


31.0
31.0, Typ
Spillway
Bulkhead gates storage
Cooling water pump room
Up
El. 410.5
Water treatment room
Contr jt, typ
℄ Units
60.0
Unwatering pumps
Drainage pumps
Turbine pit access, typ
30.80
37.45
Htg & vent fan room
Compressor room
Dn
Up
Elevator shaft
Draft tube gate service chamber
Draft tube access shaft, typ
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
Scale 0 5 10 15 20 25 Metres

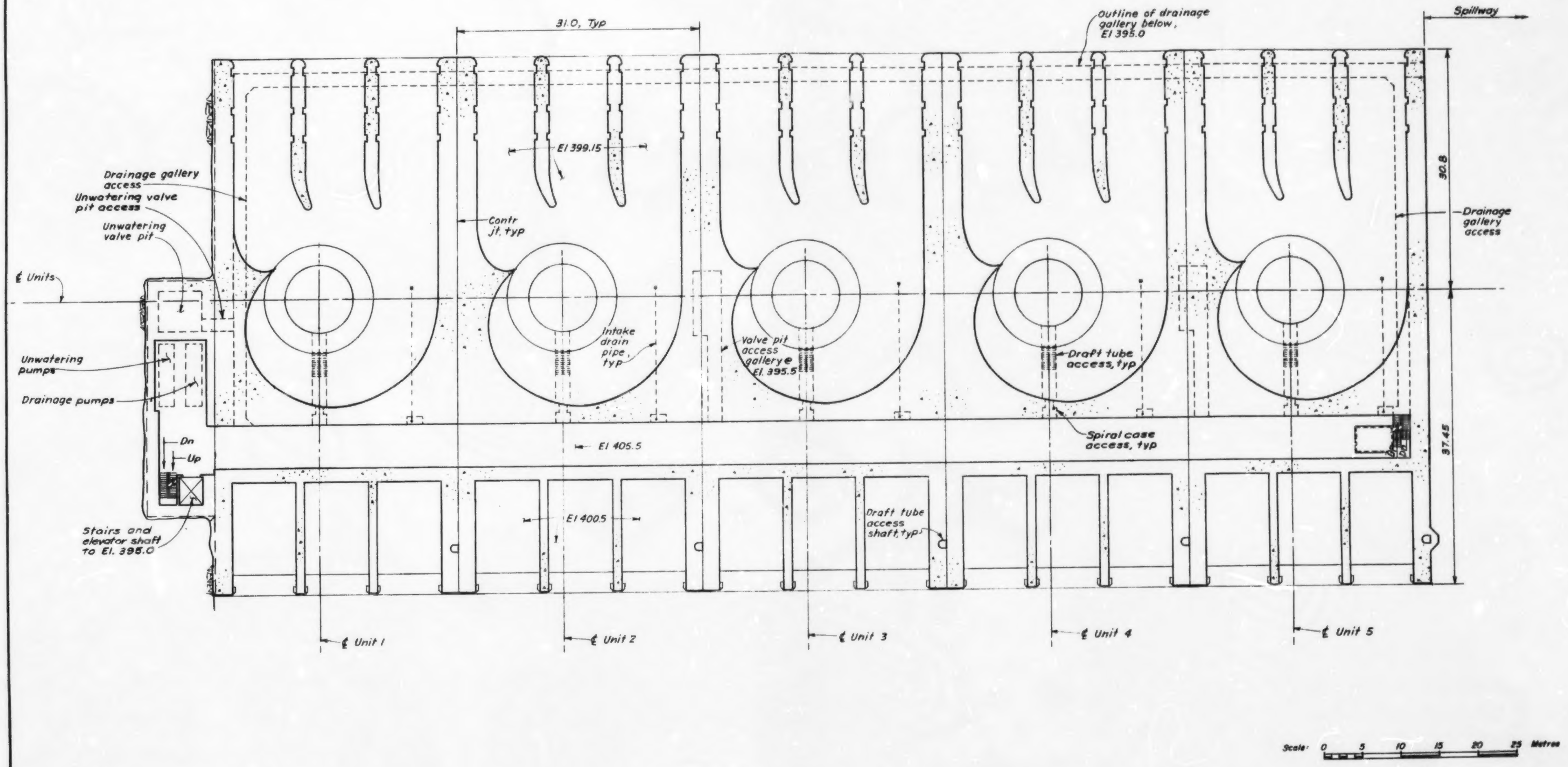
BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

POWERHOUSE
TURBINE FLOOR
PLAN EL 410.50

DATE MAY 1983	FIG. 9-9
DWN SMD	DWG No 218-C14-D636

REPORT No H1617


31.0, Typ
Outline of drainage gallery below, El 395.0
Spillway
El 399.15
30.8
37.45
Drainage gallery access
Unwatering valve pit access
Unwatering valve pit
Contr jt, typ
Drainage gallery access
℄ Units
Intake drain pipe, typ
Valve pit access gallery @ El. 395.5
Draft tube access, typ
Unwatering pumps
Drainage pumps
Spiral case access, typ
El 405.5
Dn
Up
El 400.5
Draft tube access shaft, typ
Stairs and elevator shaft to El. 395.0
℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
Scale: 0 5 10 15 20 25 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT
POWERHOUSE
℄ DISTRIBUTOR
PLAN EL 406.00

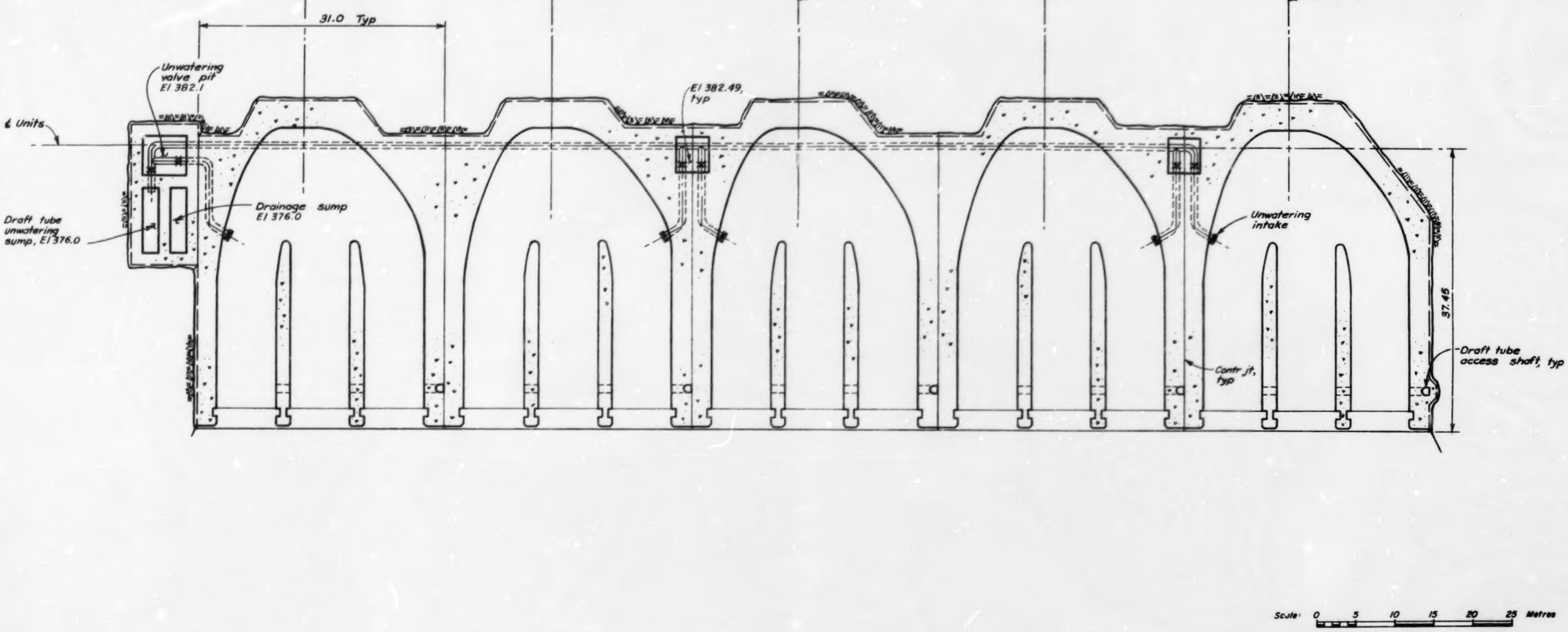
DATE MAY 1983

FIG. 9-10

DWN PK/MWB

DWG No. 218-C14-D637

REPORT No. H1617


℄ Unit 1
℄ Unit 2
℄ Unit 3
℄ Unit 4
℄ Unit 5
31.0 Typ
Unwatering valve pit El 382.1
℄ Units
El 382.49, typ
Draft tube unwatering sump, El 376.0
Drainage sump El 376.0
Unwatering intake
37.45
Contr jt, typ
Draft tube access shaft, typ
Scale 0 5 10 15 20 25 Metres

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

POWERHOUSE
DRAFT TUBE
PLAN EL 388.50

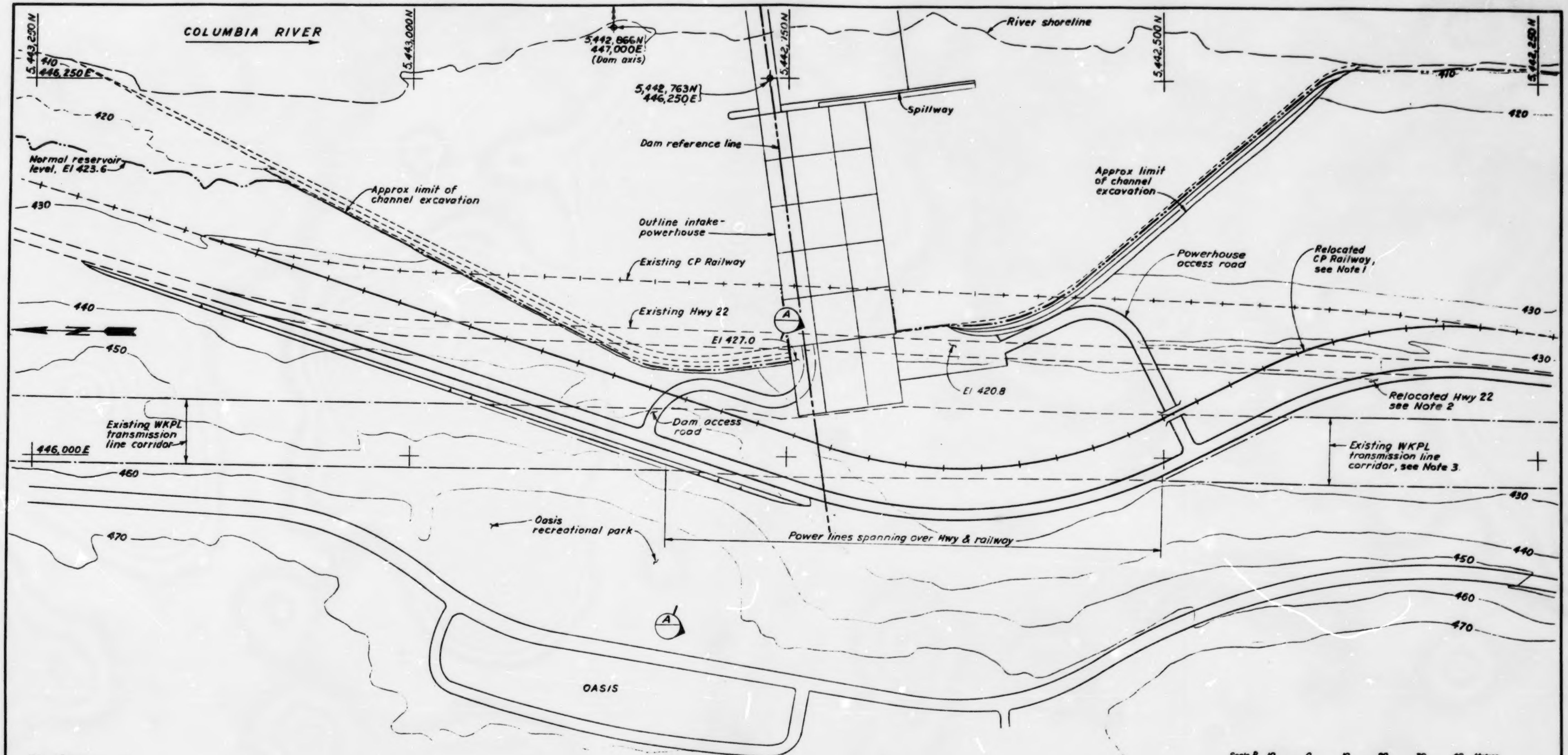
DATE MAY 1983

FIG. 9-11

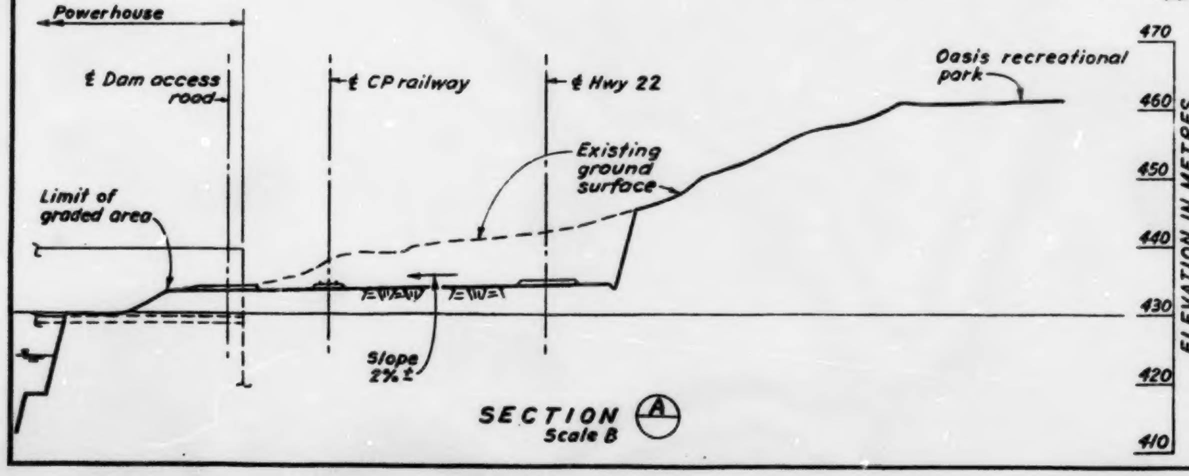
DWN SMD

DWG No. 218-C14-D638

REPORT No. H1617


COLUMBIA RIVER
5,443,250 N
410
446,250 E
5,443,000 N
5,442,866 N
447,000 E
(Dam axis)
5,442,763 N
446,250 E
5,442,750 N
River shoreline
5,442,500 N
5,442,250 N
410
420
Spillway
Dam reference line
Normal reservoir level, El 423.6
Approx limit of channel excavation
430
Outline intake-powerhouse
Approx limit of channel excavation
Existing CP Railway
Powerhouse access road
Relocated CP Railway, see Note 1
440
Existing Hwy 22
A
430
N
450
El 427.0
El 420.8
430
Relocated Hwy 22 see Note 2
Existing WKPL transmission line corridor
Dam access road
446,000 E
Existing WKPL transmission line corridor, see Note 3
460
430
Oasis recreational park
470
Power lines spanning over Hwy & railway
440
450
460
470
A
OASIS
Scale B. 10 0 10 20 30 40 Metres
Scale A: 25 0 25 50 75 100 Metres

PLAN
Scale A


Powerhouse
℄ Dam access road
℄ CP railway
℄ Hwy 22
Oasis recreational park
Existing ground surface
Limit of graded area
Slope 2% ±
470
460
450
440
430
420
410
ELEVATION IN METRES

SECTION A
Scale B

NOTES:

1. DESIGN CRITERIA FOR RAILWAY RELOCATION

HORIZONTAL CURVES	6° MAX.
STRAIGHTS BETWEEN CURVE TANGENTS	75 m MIN.
MAX. GRADE ON STRAIGHTS	1%
MAX. GRADE ON CURVE	1% LESS 0.04% FOR EACH DEGREE OF CURVE

2. RELOCATED HIGHWAY TO BE DESIGNED IN ACCORDANCE WITH THE B.C. PROVINCIAL MINISTRY OF TRANSPORTATION AND HIGHWAYS PUBLICATION "HIGHWAY DESIGN - MANUAL OF STANDARDS AND INSTRUCTIONS" FOR 80 km/h MINIMUM.

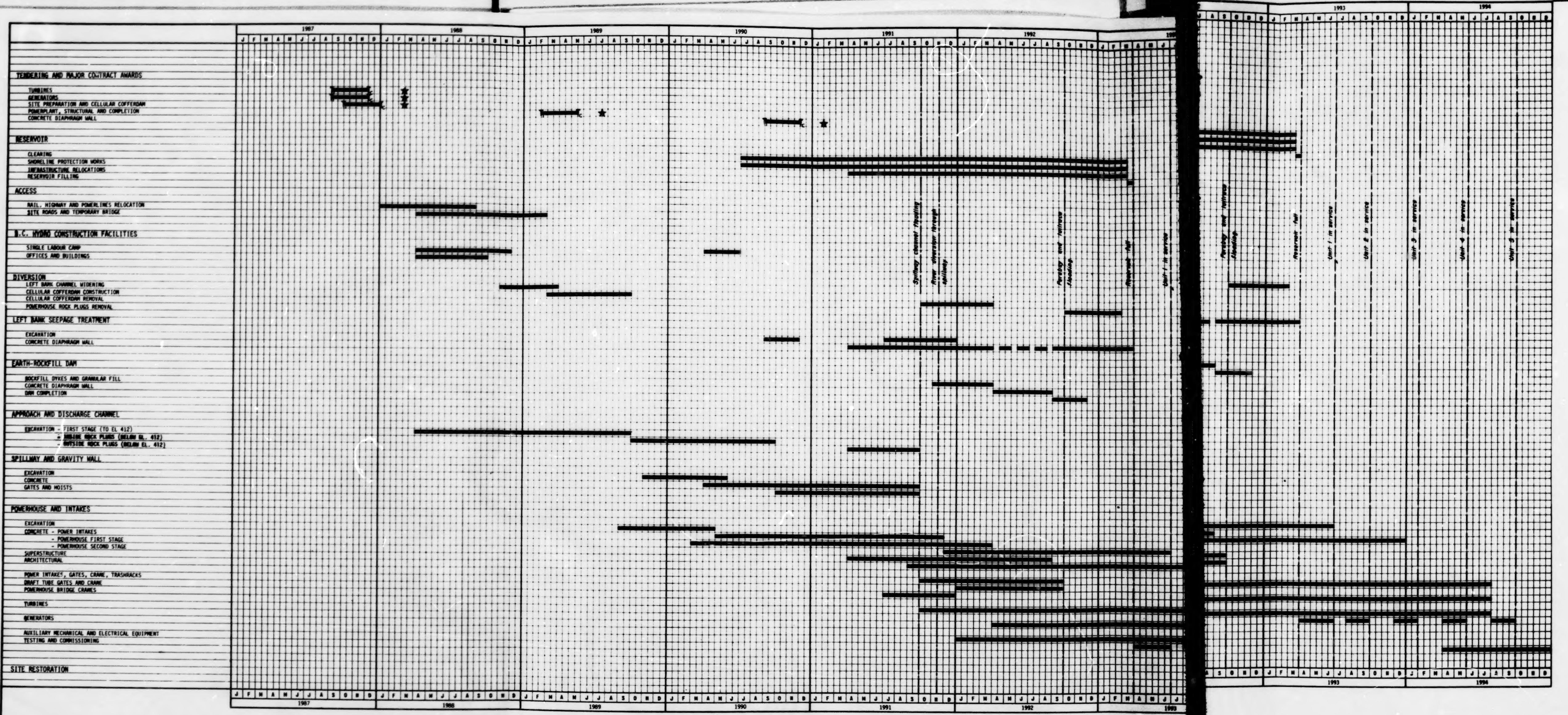
3. WKPL RELOCATED TEMPORARILY AND RE-ESTABLISHED IN EXISTING CORRIDOR FOLLOWING RELOCATION OF RAILWAY AND HIGHWAY.

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT

HIGHWAY, RAILWAY & TRANSMISSION
LINE RELOCATION

DATE MAY 1983	FIG. 9-12
DWN MWB	DWG No. 218-C14-D642



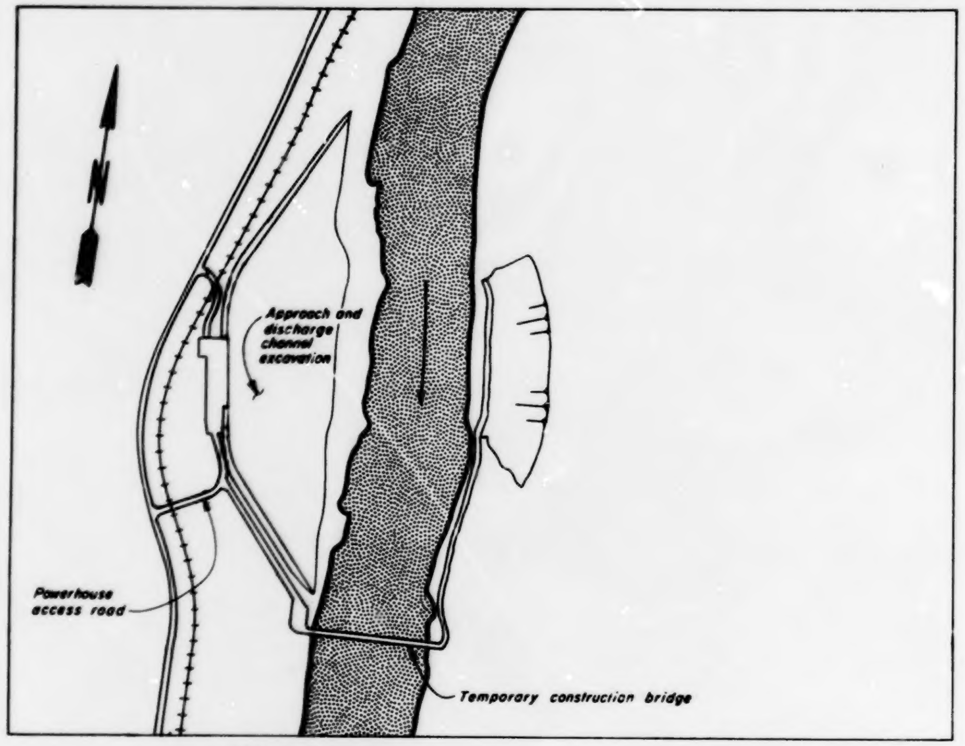
NOTES::

1. PROJECT APPROVAL REQUIRED AT LEAST SEVEN MONTHS PRIOR TO AWARD OF TURBINE CONTRACT.

LEGEND:

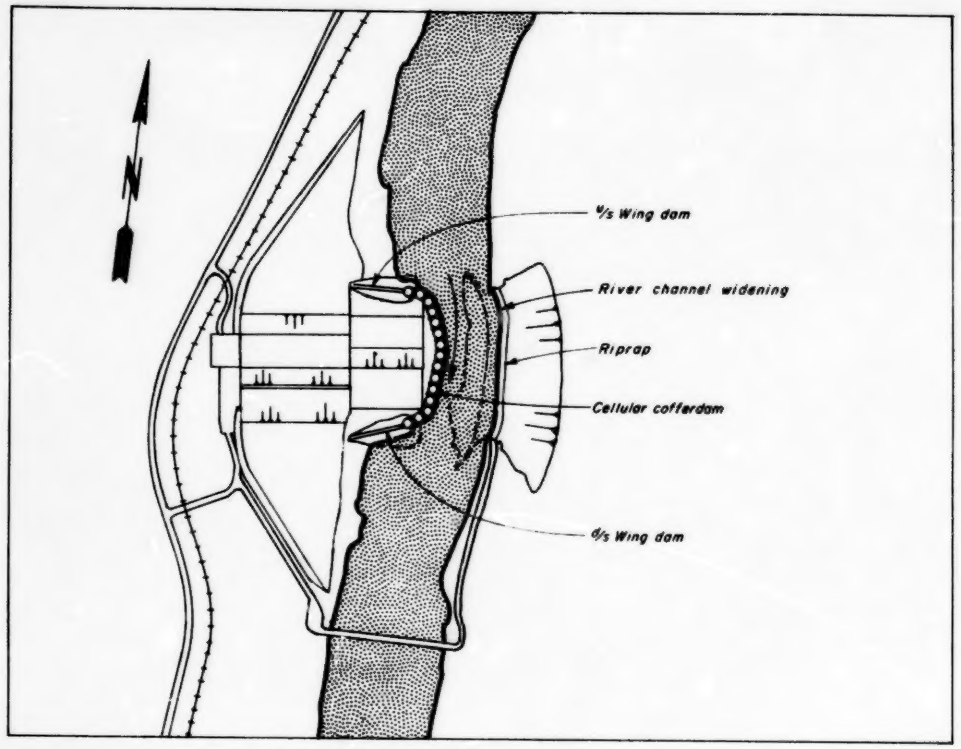
T_I TENDER ISSUE
T_C TENDER CLOSE
★ CONTRACT AWARD

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY	
COLUMBIA RIVER - MURPHY CREEK PROJECT CONSTRUCTION SCHEDULE	
DATE MAY 1983	FIG. 10-1
DWN SMD	DWG No 218-C14-X639


N
Approach and discharge channel excavation
Powerhouse access road
Temporary construction bridge

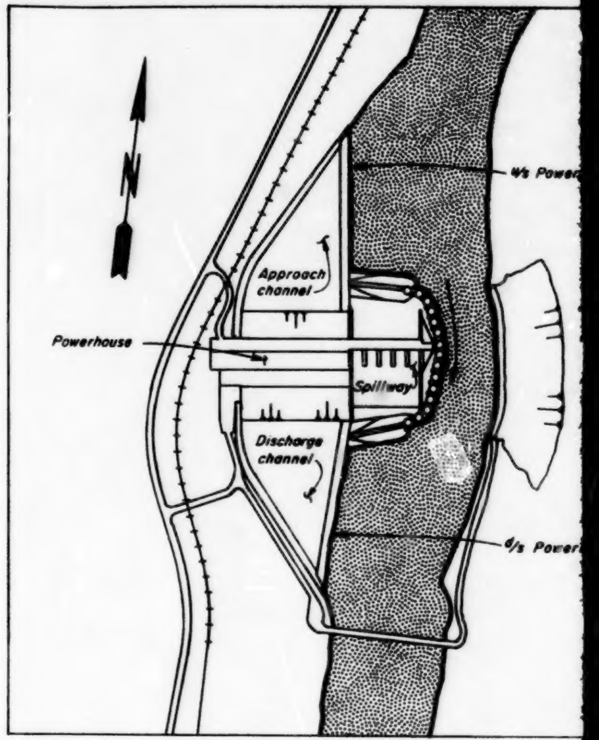
PHASE A SITE PREPARATION

-RELOCATE TRANSMISSION LINES, HIGHWAY, RAILWAY
-CONSTRUCT SITE ACCESS ROADS, TEMPORARY BRIDGE AND PREPARE WORK AREAS
-COMMENCE APPROACH AND DISCHARGE CHANNEL EXCAVATION TO EL. 412
-COMMENCE LEFT BANK RIVER CHANNEL WIDENING


N
u/s Wing dam
River channel widening
Riprap
Cellular cofferdam
d/s Wing dam

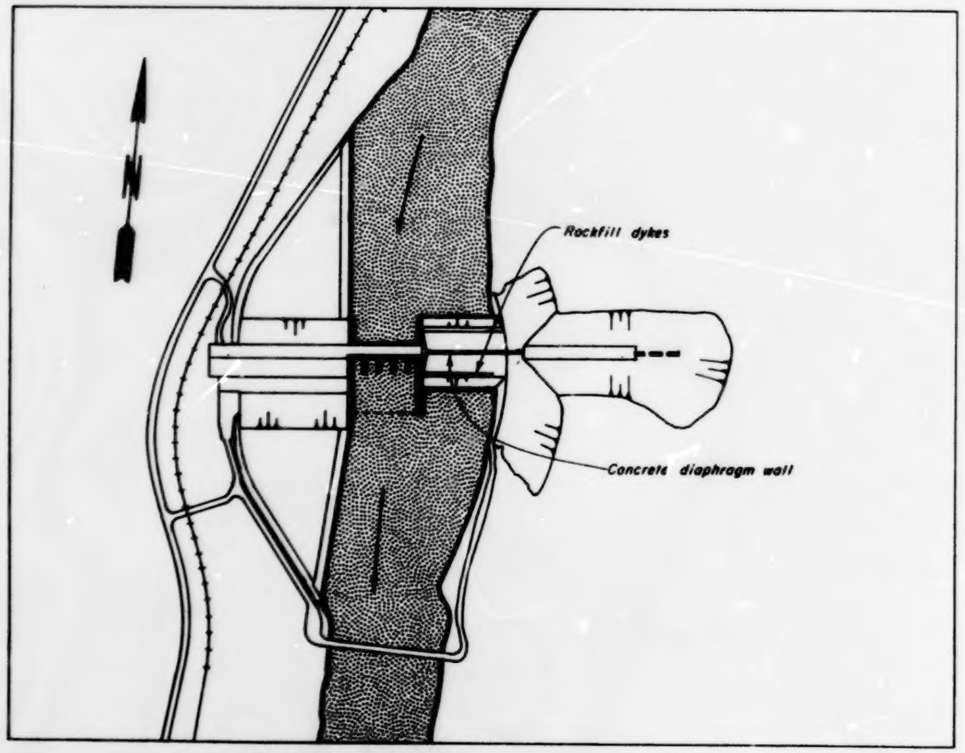
PHASE B FIRST STAGE DIVERSION

-COMPLETE RIVER CHANNEL WIDENING AND PLACE RIPRAP PROTECTION
-CONSTRUCT CELLULAR COFFERDAM AND WING DAMS
-EXCAVATE POWERHOUSE AND SPILLWAY
-CONTINUE APPROACH AND DISCHARGE CHANNEL EXCAVATION


N
u/s Powe
Approach channel
Powerhouse
Spillway
Discharge channel
d/s Powe

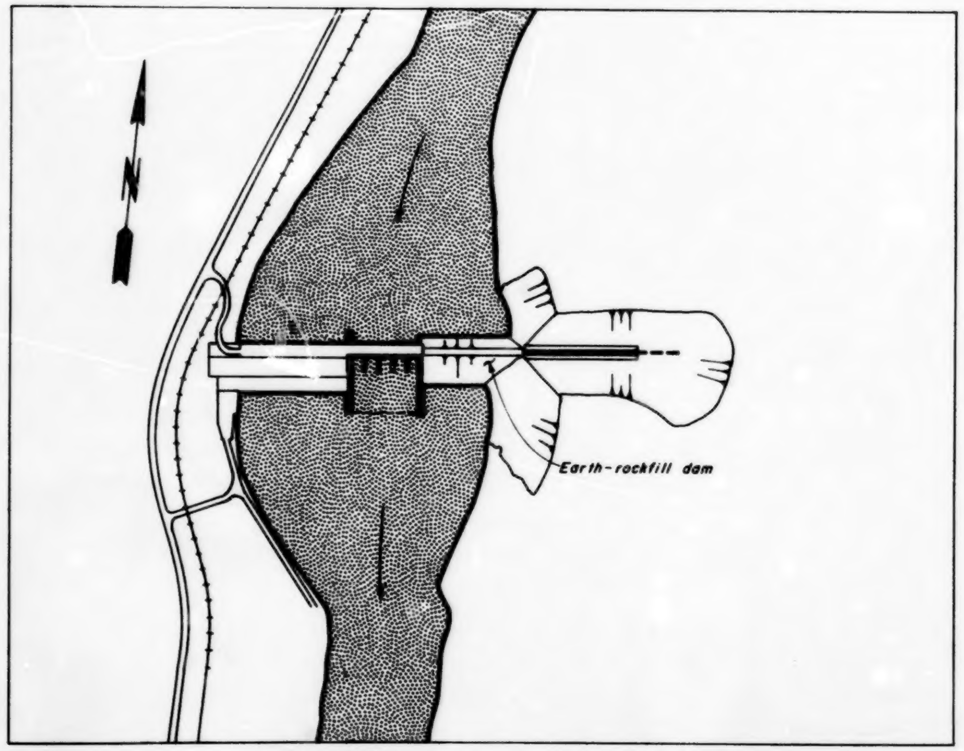
PHASE C POWERPLANT CONSTRUCTION

-CONSTRUCT POWERHOUSE AND SPILLWAY STRUCT
-INSTALL AND TEST SPILLWAY GATES
-EXCAVATE RIGHT BANK RIVERBED AND ROCK M
-COMPLETE APPROACH AND DISCHARGE CHANNEL
-COMMENCE LEFT BANK OVERBURDEN EXCAVATION
DIAPHRAGM WALL CONSTRUCTION


N
Rockfill dykes
Concrete diaphragm wall

PHASE D SECOND STAGE DIVERSION

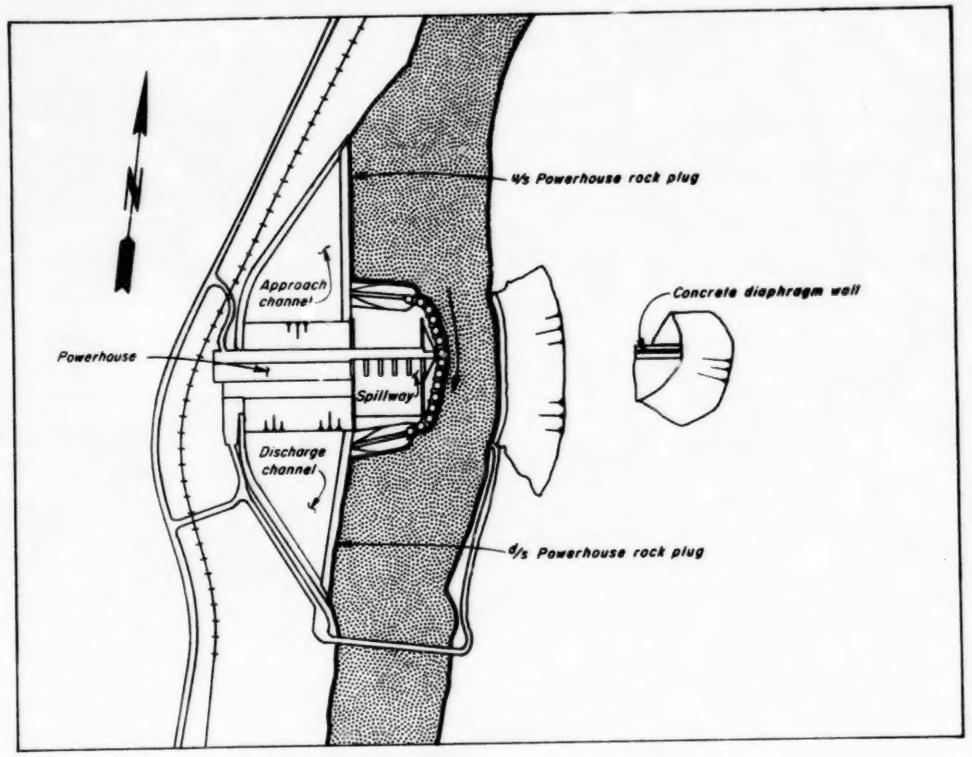
-REMOVE WING DAMS & CELLULAR COFFERDAM
-CONSTRUCT DAM ROCKFILL DYKES TO DIVERT RIVER THROUGH SPILLWAY
-PLACE GRANULAR FILL BETWEEN DYKES
-CONSTRUCT PORTION OF CONCRETE DIAPHRAGM WALL BELOW EMBANKMENT DAM
-COMPLETE LEFT BANK EXCAVATION FOR CONCRETE DIAPHRAGM WALL
-CONTINUE POWERHOUSE CONSTRUCTION AND INSTALLATION OF EQUIPMENT


N
Earth-rockfill dam

PHASE E COMPLETION

-COMPLETE EARTH-ROCKFILL DAM CONSTRUCTION
-COMPLETE LEFT BANK CONCRETE DIAPHRAGM WALL CONSTRUCTION
-INSTALL INTAKE AND DRAFT TUBE GATES
-REMOVE POWERHOUSE ROCK PLUGS
-FILL RESERVOIR
-COMPLETE INSTALLATION AND TESTING OF EQUIPMENT
-REMOVE CONSTRUCTION FACILITIES AND RESTORE SITE

BR
COLUMB
CONST
DIV
DATE MA
DWN JE
REPORT No.


u/s Powerhouse rock plug
Concrete diaphragm wall
Approach channel
Powerhouse
Spillway
Discharge channel
d/s Powerhouse rock plug

PHASE C POWERPLANT CONSTRUCTION

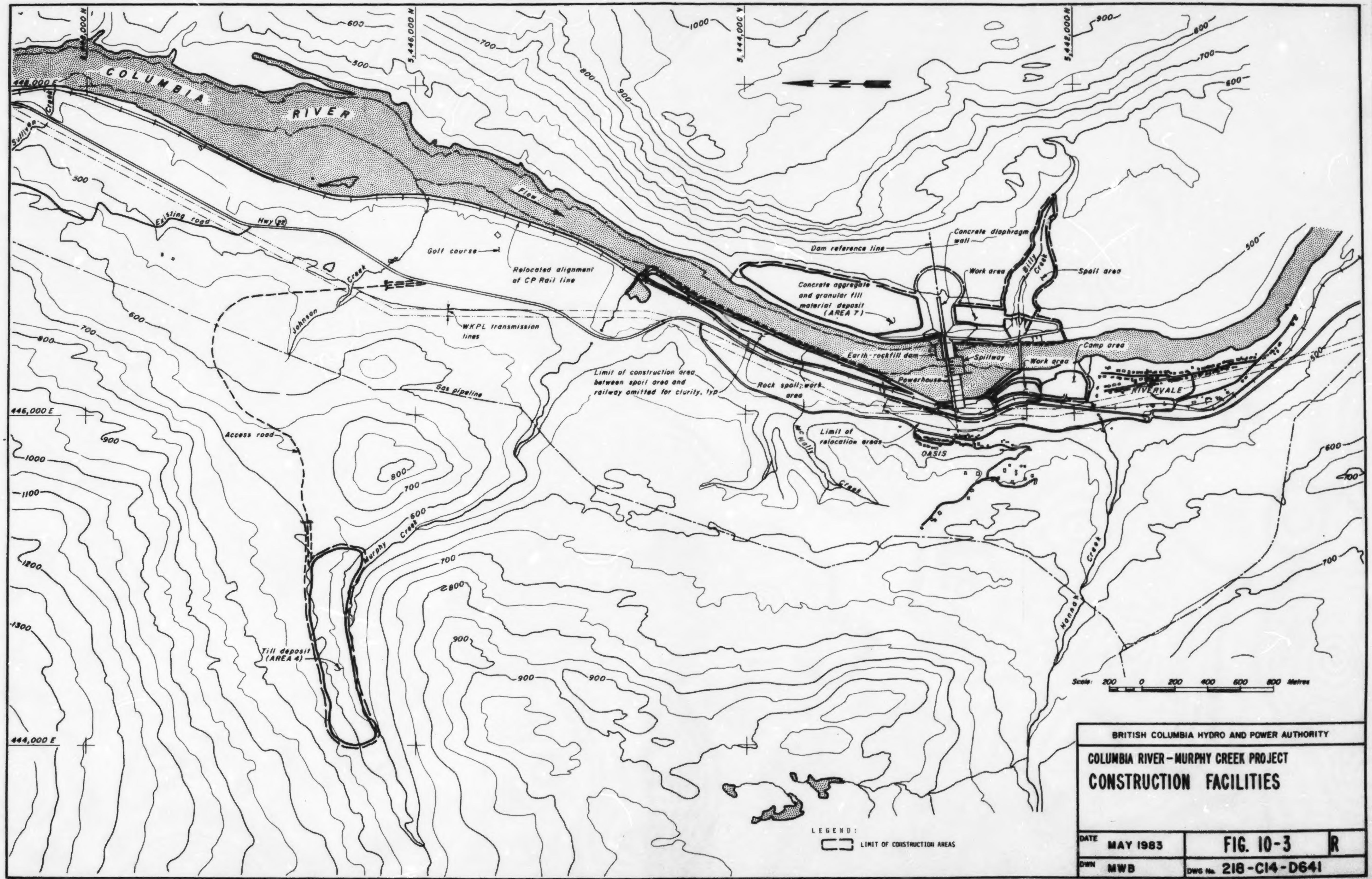
-CONSTRUCT POWERHOUSE AND SPILLWAY STRUCTURES
-INSTALL AND TEST SPILLWAY GATES
-EXCAVATE RIGHT BANK RIVERBED AND ROCK MATERIALS (OUTSIDE ROCK PLUGS)
-COMPLETE APPROACH AND DISCHARGE CHANNEL EXCAVATION (INSIDE ROCK PLUGS)
-COMMENCE LEFT BANK OVERBURDEN EXCAVATION AND CONCRETE
DIAPHRAGM WALL CONSTRUCTION

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER - MURPHY CREEK PROJECT
CONSTRUCTION AND RIVER
DIVERSION SEQUENCE

DATE MAY 1983
FIG. 10-2
DWN JE
DWG No. 218 - C14 - U640

REPORT No. H1617


COLUMBIA RIVER
Flow
5,448,000 N
5,446,000 N
5,444,000 N
5,442,000 N
448,000 E
446,000 E
444,000 E
Sullivan Creek
Existing road
Hwy 22
Golf course
Relocated alignment of CP Rail line
Johnson Creek
WKPL transmission lines
Gas pipeline
Access road
Murphy Creek
Till deposit (AREA 4)
Limit of construction area between spoil area and railway omitted for clarity, typ
Rock spoil; work area
McNally Creek
Limit of relocation areas
OASIS
Dam reference line
Concrete diaphragm wall
Concrete aggregate and granular fill material deposit (AREA 7)
Work area
Billy Creek
Spoil area
Earth-rockfill dam
Spillway
Powerhouse
Work area
Camp area
RIVERVALE
Hannah Creek
300
500
600
700
800
900
1000
1100
1200
1300
Scale: 200 0 200 400 600 800 Metres

LEGEND:

⊏⊐ LIMIT OF CONSTRUCTION AREAS

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY

COLUMBIA RIVER – MURPHY CREEK PROJECT

CONSTRUCTION FACILITIES

DATE MAY 1983

FIG. 10-3

R

DWN MWB

DWG No. 218-C14-D641

REPORT No. H1617

APPENDIX A

APPENDIX A

MURPHY CREEK PROJECT

PRELIMINARY DESIGN GUIDELINES

APPENDIX A

MURPHY CREEK PROJECT
PRELIMINARY DESIGN GUIDELINES

A.1 BASIC PROJECT DATA (for 5-Unit, 300 MW Kaplan powerplant)

(a) Discharges

Average River Flow (1940-1975)	2 050 m³/s
Flow per unit (maximum)	524 m³/s
Diversion Design Flood	6 310 m³/s
200-year Flood	7 590 m³/s
Probable Maximum Flood (PMF)	13 100 m³/s

(b) Water Levels

Headwater	Project Site	Castlegar
Discharge 2050 m³/s	423.6 m	424.0 m
Discharge 2620m³/s	423.3 m	424.0 m
200-year Flood	418.2 m	425.8 m
PMF	424.2 m	431.0 m
Diversion Design Flood (1st stage)	415.7 m	424.2 m
Diversion Design Flood (2nd stage)	416.3 m	424.2 m

Tailwater	
Discharge 2050 m³/s	408.8 m
Discharge 2620 m³/s	409.7 m
Discharge 425 m³/s	405.3 m
200-year Flood	415.6 m
PMF	419.9 m
Diversion Design Flood	414.3 m

A.2 LOADS (for gravity structures)

(a) Mass of Materials

Granular Fill	2 160 kg/m³
Concrete	2 400 kg/m³
Foundation Rock	2 640 kg/m³

(b) Hydrostatic and Hydrodynamic Pressure - As applicable. (Hydrodynamic pressure acts above level of rock)

(c) Uplift

The uplift pressures are based on the following assumptions:

1. Uplift and internal hydraulic pressures are not affected by earthquake forces and are applied over 100 percent of the base area.

2. Uplift does not operate in a crack formed during an earthquake.

3. With drains operational: full headwater pressure at the upstream edge of base, then varying linearly to a pressure at the drains equal to tailwater plus one-third of the headwater-tailwater difference, and then varying linearly to tailwater at the downstream edge of base (for gravity structures except powerhouse).

4. With drains non-operational: full headwater pressure at the upstream edge of base varying linearly to tailwater at downstream edge of base (for powerhouse).

(d) Earthquake Loads

Horizontal acceleration 0.1 g with a vertical acceleration of 0.05 g acting simultaneously.

(e) Ice Load

A force of 145 kN per linear metre is assumed to act at 0.3 m below the reservoir surface on all water-retaining structures. For spillway piers this force is assumed to be 290 kN per linear metre of ice contact surface and acting at 0.3 m below the reservoir surface. There is assumed to be no ice load on the spillway gates.

A.3 LOAD COMBINATIONS

Three load combinations have been considered in the stability analyses as follows:

1. Usual Load Combination - includes gravity, hydrostatic, ice and uplift force with headwater at El. 424 m and tailwater at El. 408.8 m.

2. Unusual Load Combination - includes gravity, hydrostatic and uplift forces based on PMF conditions with headwater at El. 424.2 m and tailwater at El. 419.9 m.

3. Extreme Load Combination - includes water levels and all the loads of the usual load combination plus the hydrodynamic and inertia loads generated by the design earthquake.

A.4 MINIMUM REQUIRED OVERTURNING, FLOATATION AND SLIDING FACTORS OF SAFETY

(a) Concrete/Rock Interface and Foundation Rock

	Allowable Concrete Stresses		Minimum Factors of Safety	
	Tensile	Compression[*1] (kPa)	Floatation[*2]	Sliding (Friction-Only Analysis)[*3]
Usual Load	0	9 000	1.2	1.5
Unusual Load	0	13 500	1.2	1.3
Extreme Load	Crack allowed tension zone neglected	27 000	1.1	>1.0

Notes

[*1] Concrete 1-year compressive strength of 27 000 kPa.

[*2] Floatation factor of safety during construction = 1.1.

[*3] Foundation Rock, coefficient of friction = 1.2 (50°) at concrete/rock interface

= 1.0 (45°) for potential sliding planes in the foundation

(b) Concrete Structures

Allowable compression stress: (same as A.4 (a) above)

Allowable tension stress (with uplift):

Usual loading	0
Unusual loading	700 kPa
Extreme loading	1 400 kPa (crack allowed; tension zone neglected)

Sliding stability:
$\phi = 1.0$ (45°) and
$C = 0.125 \times$ concrete 1 year compressive strength along horizontal construction joints

Minimum acceptable shear friction factors (SFF):

Usual loading	3
Unusual loading	2
Extreme loading	1